DEPARTMENT OF THE ARMY

CORPS OF ENGINEERS

MISSISSIPPI RIVER COMMISSION

CATFISH POINT CONTROL STRUCTURE FOUNDATION INVESTIGATION

TECHNICAL MEMORANDUM NO. 3-249

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

JANUARY 1948

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CATFISH POINT CONTROL STRUCTURE
FOUNDATION INVESTIGATION

Location and Description of Project

1. The proposed Catfish Point control structure is on the Mermentau River at Grand Lake. Geopgraphically, it is about 30 miles southeast of Lake Charles, Louisiana. The structure will provide protection against high tides and salt water intrusion. A new and deeper channel will facilitate the passage of flood flows down the Mermentau River. If navigation traffic is of sufficient volume, one gate may be developed into a lock, to facilitate the movement of navigation during various river tides and flood stages.

2. The main control structure (plate 4) consists of two sector gates, 2 and 3, with a free flow width of 56 ft each. The bottom of the channel and top of the base slab are at elev -15.0. The bottom of the base slab for the sector gates is at elev -19.0. A 4-in. concrete stabilization course and a 12-in. filter make the bottom of the excavation at elev -20.3. The top of the structure is at elev +6.0, the same elevation as the top of the tie-in dike. The base slab for the sector gates is to be of reinforced, monolithic construction. The only joint will be between the two gate structures.

3. Gate 1, which may in the future be developed into a lock, also has a clear width of 56 ft (see plate 4). The bottom of the channel and top of base slab are at elev -10. The bottom of the base slab is at

elev -13.5 with a 4-in. concrete stabilization course and 12-in. filter below. The base slab is to be of reinforced, monolithic construction.

4. The dike will be tied into the structures with two sheet pile walls, 5 ft apart, backfilled with earth. The main control structure and the lock gate structure will be connected by a similar wall (see plate 4). There is a short section of levee perpendicular to the connecting wall between the control structure and lock gate which will become part of one side of the lock when it is built.

Authority and Purpose of Investigation

5. The investigation of the foundation for this structure was authorized in a Job Estimate Cost Summary (Form 18) prepared by the Mississippi River Commission, dated 23 June 1947.

6. The following problems were considered in the investigation and are treated in this report:

- a. Excavation (stability of slopes and dewatering).
- b. Determination of most suitable type of foundation, pile or slab.
- c. Settlement of the structures.
- d. Underseepage and filter design.
- e. Location of spoil banks.
- f. Fender piles.

Field Exploration

7. The field exploration consisted of three undisturbed borings and twelve general sample borings. The undisturbed borings were made with a 5-in. Shelby tube sampler along the centerline of the structures and to depths of 83, 65, and 65 ft, borings 1, 2, and 3, respectively. Three general sample borings were made to depths of 37 ft along the proposed channel connecting the structure to Grand Lake. Nine additional shallow auger borings were made to locate the upper surface of the Pleistocene Deltaic Plain deposit, which underlies the structure at this site. The locations of all borings are shown on plate 1. The logs of the borings are plotted on plate 2.

Description of Soil Conditions

8. The preliminary report on the geological investigation of this site by Dr. H. N. Fisk[1] classified the soft gray top strata (about 12 ft thick) as Recent marsh deposits, while the drier, more firm, oxidized material below it was termed Pleistocene Deltaic Plain deposits. The base of the control structure will be about 10 ft below the surface of the Pleistocene deposit. However, the report interpreted the borings then available as indicating the possibility of an erosional valley in the Pleistocene surface, filled with soft Recent alluvium under the

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1 Preliminary Report "Geological Investigation of the Grand Cheniere and Catfish Point Lock and Control Structure Sites, Mermentau Basin, La." by Dr. H. N. Fisk, Consultant.

lock gate. The additional nine borings made to locate the Pleistocene surface did not disclose any such valley immediately beneath the base of the structure. The base of the lock gate (elev -13.5) will be below the Pleistocene surface indicated in all borings (approximately elev -10). Although the boring logs show more or less definite strata of certain types of soil, these strata are further stratified. For example, lenses of clay silt were frequently noted in samples from the silty clay and clay strata. The ground-water table was just below the surface of the ground at the time the borings were made.

9. The elevation of the ground surface is about +2 ft MLG. The generalized profiles on plate 3 based on the boring logs shown on plate 2 indicate that the soil conditions may be generalized as follows:

Elev - Ft MLG	Description
Recent deposits:	
+2 to -10	Silty clay, gray, soft
Pleistocene deposits:	
-10 to -19	Silty clay with some sandy clay and clay sand, tan and gray, firm; some shells.
-19 to -29	Clay and silty clay, red and gray, soft to firm.
-29 to -36	Clay with many sandy silt lenses, tan and gray, stiff.
-36 to -38	Shells, clay and sandy silt with shells.
-38 to -47	Clay and silty clay, gray, stiff; some shells.
-47 to -55	Clay silt, gray, soft to firm; some shells.
-55 to -71	Clay and silty clay, tan and gray, firm.
-71 to -76	Silty sand and clay sand, gray.
Below -76	Sand, fine, gray.

Laboratory Tests

General

10. Classification tests were made on selected samples from all borings. These included water content, mechanical analyses, and Atterberg limits. Shear strength tests (unconfined and quick triaxial compression tests) and consolidation tests were made on samples from the undisturbed borings. The results of the laboratory tests are summarized in table 1. The shear strengths of the specimens tested are shown on the boring logs (plate 2) with the water contents of the test specimens.

Mechanical analyses and Atterberg limits

11. Typical grain-size curves of the various soils are shown on plates 5 through 8. Most of the mechanical analyses were performed to check the visual classification of the samples and are therefore not included in table 1. Those analyses made on samples on which shear and consolidation tests were made are included in table 1.

12. No Atterberg limit tests were made on samples from the Recent stratum, as the base of the structures will be below this stratum. Two Atterberg limit tests made on samples from the upper portion of the Pleistocene deposit (elev -10 to elev -19) gave the following average results: Liquid Limit (LL) = 50, Plastic Limit (PL) = 14, Plasticity Index (PI) = 36. Five tests on samples from the deeper Pleistocene strata gave the following range and average of Atterberg limits:

	Range	Average
Liquid Limit (LL)	63-88	75
Plastic Limit (PL)	20-27	23
Plasticity Index (PI)	43-62	51

TABLE 1

Waterways Experiment Station
Catfish Point Control Structure

SUMMARY OF TEST DATA

Sample No.	Sample Depth Ft	Sample Elev MGL	Classification	Mechanical Analysis % Sand	Mechanical Analysis % Silt	Mechanical Analysis % Clay	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Sp. Gr.	Nat. w - %	Shear Test Type	Shear Test w - %	Shear Test d Lb/Cu Ft	Shear Test ø°	Shear Test c T/Sq Ft	Consolidation Test C_c	Consolidation Test P_c T/Sq Ft	Consolidation Test P_o T/Sq Ft	Consolidation Test $t_{50\%}$ Min.	Consolidation Test w - %
Boring 1 - Gr.El. + 2.8																					
7	6.8- 7.8	-4.5	Silty clay, grey, soft	23	41	36	-	-	-	-	51	QT	51	69	1°	0.18	-	-	-	-	-
10	10.0-10.9	-7.6	Silty clay, tan & grey, soft	-	-	-	-	-	-	-	27	UC	28	94	-	0.19	-	-	-	-	-
13	13.1-13.9	-11.8	Silty clay, grey-brown, stiff	29	33	38	48	12	36	2.69	32	UC	22	104	-	0.53	0.20	1.4	0.43	60	23
												QT	24	100	1°	0.58					
21	21.8-22.7	-19.5	Clay, red & grey, firm	-	-	-	-	-	-	-	37	UC	35	86	-	0.44	-	-	-	-	-
22	22.7-23.8	-20.5	Clay, red & grey, firm	-	-	-	-	-	-	-	36	UC	34	87	-	0.37	-	-	-	-	-
24	24.9-25.8	-22.6	Clay, grey-brown, stiff	-	-	-	-	-	-	-	43	UC	33	87	-	0.64	-	-	-	-	-
25	25.8-26.8	-23.6	Clay, tan & grey, firm	9	8	83	86	27	59	2.71	36	UC	38	83		0.42	0.32	1.8	0.78	44	41
												QT	38	83	2°	0.68					
31	32.2-33.1	-29.9	Clay, brown, stiff, w/thin lenses of	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
			Clay silt	-	-	-	-	-	-	-	34	UC	34	86	-	0.97	-	-	-	-	-
32	33.1-34.3	-30.9	Clay, tan & grey, stiff, w/thin lenses of sandy silt	-	-	-	-	-	-	-	36	UC	33	86	-	0.70	-	-	-	-	-
34	35.4-36.4	-33.1	Clay, tan & grey, stiff, w/thin lenses of sandy silt	7	25	68	68	22	46	2.71	41	-	-	-	-	-	0.35	3.2	1.02	20	37
35	36.4-37.3	-34.1	Clay, tan & grey, stiff, " " "	-	-	-	-	-	-	-	48	UC	41	80	-	0.87	-	-	-	-	-
42	44.2-45.1	-41.9	Clay, grey, stiff, w/shells	-	-	-	-	-	-	-	64	UC	46	73	-	0.65	-	-	-	-	-
43	45.1-46.5	-42.5	Clay, grey, stiff w/organic matter & concretions	-	-	-	-	-	-	-	63	UC	72	55	-	0.60	-	-	-	-	-
53	55.8-56.7	-53.5	Clay silt, grey, firm	-	-	-	-	-	-	-	24	UC	25	97	-	0.47	-	-	-	-	-
64	67.3-68.3	-65.8	Clay, grey-brown, firm, fissured	-	-	-	-	-	-	-	36	UC*	33	87	-	0.24	-	-	-	-	-
65	68.3-69.2	-66.8	Clay, grey-brown, firm	-	-	-	-	-	-	-	36	UC	38	81	-	0.44	-	-	-	-	-
Boring 2 - Gr.El. + 2.5																					
1	0 - 1.5	+ 1.3	Clay silt, grey, soft w/much humus	-	-	-	-	-	-	-	44	UC	42	76	-	0.19	-	-	-	-	-
6	5.9-8.4	-4.7	Silty clay, grey, soft	20	35	45	-	-	-	-	47	QT	40	82	2°	0.17	-	-	-	-	-
11	16.0-17.0	-14.0	Silty clay, tan & grey, firm, with concretions	-	-	-	52	17	35	-	22	UC	23	102	-	0.63	-	-	-	-	-
13	18.0-19.0	-16.0	Silty clay, tan & grey, firm	21	35	44	-	-	-	-	21	UC	24	101	-	0.34	-	-	-	-	-
14	19.0-20.1	-17.0	Silty clay, tan & grey, stiff	-	-	-	-	-	-	-	27	UC	21	103	-	0.52	-	-	-	-	-
22	27.5-28.5	-25.5	Clay, red & grey, firm	6	9	85	-	-	-	-	41	UC	43	78	-	0.30	-	-	-	-	-
26	31.7-32.7	-29.7	Clay, tan & grey, stiff	8	22	70	68	21	47	2.71	37	-	-	-	-	-	0.48	4.5	0.80	18	34
31	36.9-38.2	-35.0	Clay, tan & grey, stiff, w/sandy silt lenses	4	14	82	-	-	-	-	44	UC	41	80	-	0.63	-	-	-	-	-
36	44.3-45.3	-42.3	Clay, grey, stiff	-	-	-	-	-	-	-	61	UC	34	86	-	0.72	-	-	-	-	-
38	46.0-47.0	-44.0	Clay, grey, firm	-	-	-	88	26	62	-	45	UC	48	75	-	0.43	-	-	-	-	-
41	50.3-51.3	-48.3	Clay silt, grey, stiff	8	64	28	-	-	-	-	26	QT	25	97	4°	0.55	-	-	-	-	-
44	53.4-54.2	-51.3	Clay silt, grey, stiff	-	-	-	-	-	-	-	27	UC	26	98		0.51	-	-	-	-	-
Boring 3 - Gr.El. + 1.9																					
4	5.6- 6.6	- 4.2	Silty clay, tan & grey, soft	15	35	50	-	-	-	-	78	UC	59	65	-	0.13	-	-	-	-	-
8	9.8-10.8	- 8.4	Silty clay, grey, soft	24	26	50	-	-	-	-	36	UC	34	87	-	0.16	-	-	-	-	-
9	10.8-11.8	- 9.4	Silty clay, tan & grey, firm	-	-	-	-	-	-	-	34	QT	29	83	1°	0.27	-	-	-	-	-
13	15.0-16.0	-13.8	Clay, tan & grey, firm	25	27	48	-	-	-	-	26	UC	26	96	-	0.42	-	-	-	-	-
18	20.2-21.2	-18.8	Silty clay, tan & grey, firm	-	-	-	-	-	-	-	34	UC	29	92	-	0.41	-	-	-	-	-
20	22.4-23.4	-21.0	Silty clay, tan, soft	-	-	-	-	-	-	-	39	UC	33	88	-	0.19	-	-	-	-	-
23	25.4-26.6	-24.1	Clay, tan & grey, firm	6	31	63	-	-	-	-	28	UC	37	84	-	0.26	-	-	-	-	-
27	29.6-30.8	-28.3	Clay, tan & grey, firm	6	11	83	-	-	-	-	39	UC	34	81	-	0.25	-	-	-	-	-
30	32.8-33.8	-31.6	Clay, tan & grey, hard, w/sandy silt lenses	-	-	-	-	-	-	-	35	UC	33	87	-	1.11	-	-	-	-	-
41	44.4-45.4	-43.0	Clay, grey, stiff	7	13	80	-	-	-	-	57	UC	33	88	-	0.62	-	-	-	-	-
49	52.9-53.9	-52.5	Clay silt, grey, firm	8	66	26	-	-	-	-	28	UC	27	97	-	0.35	-	-	-	-	-
54	58.0-59.1	-56.6	Clay, grey, firm	-	-	-	63	20	43	2.70	30	-	-	-	-	-	0.30	4.0	1.46	53	29

NOTES:

UC - Unconfined compression test
QT - Quick triaxial compression test
Each UC test value shown is average of two tests
w% for shear and consolidation tests is water content before testing
Po computed on basis of existing water table
Average t50% for test loads of 3.2, 6.4, and 12.8 tons sq/ft.
Specimen 1.2 in. thick, drained top and bottom.
* 5-in. diameter specimen

Consolidation tests

13. Consolidation tests indicate that the foundation clays have been consolidated to pressures greater than the existing overburden (see plates 9 and 10). This overconsolidation may have been caused by alternate wetting and drying, as these materials are highly oxidized. The computed preconsolidation pressures range from 1.0 to 3.7 tons per ft greater than the existing overburden pressure. Therefore, the laboratory recompression void ratio curves were used rather than the straight-line virgin curve for settlement computations. The compression indices (C_c) for the clay and silty clay range from 0.20 to 0.48, with an average of 0.33. However, as these soils are overconsolidated, the compression index is not indicative of the compressibility of the soils in the load range imposed by this structure (see paragraphs 22 and 25).

Shear strength tests

14. All of the quick triaxial compression tests (plates 11-16 and table 1) indicated a ϕ of 4 degrees or less. Therefore, a ϕ of zero degrees was assumed for all clayey materials, since the frictional resistance contributed by the small angles of friction obtained is low in relation to the cohesive shear strength of the material. The cohesive shear strengths as determined by unconfined compression and quick triaxial tests are plotted on plate 17, which shows the variation of water content and strength with depth. The assignment of shear strengths to the various strata listed in paragraph 9 for use in stability analyses was based on this plot. The range and average of test results and the shear strength assigned to each general stratum are as follows:

Stratum Number	Elevation Ft - MLG	Range of Shear Strengths Tons/Sq Ft	Average Shear Strength Tons/Sq Ft	Design Shear Strength Tons/Sq Ft
1	+2 to -10	0.16 to 0.19	0.18	0.18
2	-10 to -19	0.27 to 0.58	0.44	0.40
3	-19 to -29	0.19 to 0.48	0.32	0.25
4	-29 to -36	0.70 to 1.10	0.91	0.65
6	-36 to -47	0.60 to 0.72	0.65	0.65
7 & 8	below -47	0.35 to 0.55	0.46	0.46

Some erratic test results were omitted in preparing the above tabulation. The values presented are based on the central 80 per cent of the test data obtained for the stratum shown.

15. The selection of the values for the second and third strata was weighted in favor of the lower strengths shown by borings 2 and 3. The values assigned to these strata are the averages of the shear strengths of samples from borings 2 and 3 (see plate 3). The high average strength of the fourth stratum was reduced to correspond more closely with the strength obtained for this stratum in boring 2 (see plate 3). All other assigned values are averages of the test results for all three borings.

Excavation

Construction slopes

16. In the stability analyses, the bottom of the excavation for the control structure was taken as elev -20. Excavation in the dry was also assumed. The stability analyses consisted of analyzing:

a. A circular arc failure within the soft upper clay stratum (bottom of arc at elev -10).

b. A circular arc failure extending down to the stiff clay stratum (elev -29).

c. A sliding wedge (method of planes) failure with the horizontal plane at elev -29.

The critical failure surfaces are shown on plate 18. The factors of safety obtained for a slope of 1 on 3 are as follows:

Analysis	Factor of Safety
Arc to elev -10	2.70
Arc to elev -29	1.50
Sliding wedge	1.36

The above factors of safety indicate that a slope of 1 on 3 will be adequate.

Channel slopes

17. The bottom of the main channel was taken at elev -15 with the ground surface at elev +2. As the channel will be dredged, the analysis was made on the assumption of water in the channel at elev +2. An analysis of a circular arc down to the stiff clay stratum (elev -29) showed a factor of safety of 4.0 for a slope of 1 on 2. A sliding wedge analysis with the horizontal plane at elev -29 gave a factor of safety of 2.8.

18. Between the lock gate and the main structure, a small levee will be built to elev +6, giving a continuous slope from elev +6 to elev -15 in the main channel. Analyses of this slope assuming the channel dewatered gave the following results for a slope of 1 on 3:

	Factor of Safety
Circular arc to -10	1.86
Circular arc to -29	1.50
Sliding wedge to -29	1.25

A slope of 1 on 3 is considered satisfactory. This levee should be constructed of material excavated from the Pleistocene deposits.

Dewatering

19. Because of the tightness of the foundation materials, it is believed that the excavation can be made in the dry without the use of well points, although adequate trenches and sumps will be required.

Foundation Design

Control structure

20. The base of the control structure is below the Pleistocene Terrace surface and will rest on 10 ft of firm clay underlain by stiff clay. The critical bearing loads which will be imposed on the foundation were furnished by the Mississippi River Commission Design Section and are as follows:

> a. An individual block of the base slab (4 ft thick, 15 by 56 ft) immediately after pouring; $q = 600$ lb per sq ft.
>
> b. A sector gate (assumed square loading of 70 by 70 ft) immediately after construction and prior to backfilling; $q = 910$ lb per sq ft.

The bearing capacities of the foundation were computed for the above loadings using Terzaghi's formulas, page 125, "Theoretical Soil Mechanics" and the indicated foundation strength of $\phi = 0^{\circ}$, cohesion (c) = 0.25 ton per sq ft. No surcharge was assumed in the computations. The results of the computations are as follows:

Loading	Bearing Capacity (Lb/Sq Ft) Type of Shear Failure General	Local	Factor of Safety General	Local
Base slab	2850	1900	4.8	3.2
Sector gate	3710	2470	4.1	2.7

From these computations it appears that the foundation has adequate bearing capacity to support the structure. However, it is pointed out that all loose and disturbed soil in the bottom of the excavation should be removed prior to placement of the filter blanket and seal course.

Lock gate

21. The base of the lock gate structure will rest on 14 ft of firm silty clay. The critical loadings for the lock gate are:

   a. An individual block of the base slab (3-1/2 ft thick, 15 by 56 ft); q = 525 lb per sq ft.

   b. The sector gate (assumed square loading 70 by 70 ft); q = 762 lb per sq ft.

For the indicated foundation strength of $\phi = 0^\circ$, c = 0.25 ton per sq ft, the bearing capacities were computed as in paragraph 20. The results of the computations are as follows:

Loading	Bearing Capacity (Lb/Sq Ft) Type of Shear Failure General	Local	Factor of Safety General	Local
Base slab	2850	1900	5.4	3.6
Sector gate	3710	2470	4.9	3.2

From the above computations the foundation appears to have adequate bearing capacity to support the structure. Any soft Recent material found beneath the base of the structures should be removed and replaced with suitable compacted material, and all loose disturbed soil in the bottom of the excavation should be removed before the filter blanket is placed.

Settlement

Control structure

22. The foundation pressure of the sector gates as furnished by the Design Section of the Mississippi River Commission is 1892 lb per sq ft for the gates closed with the water at elev 6.0 on the Gulf side and elev 0.0 on the Grand Lake side, assuming no buoyancy or uplift. When the hydrostatic pressure beneath the slab (uplift to elev 0.0) is deducted, the load is reduced to 721 lb per sq ft. As the weight of the soil excavated will amount to about 1100 lb per sq ft, the net settlement load will be negative.

23. As the connecting dikes are joined to the structure by two rows of sheet piles filled between with impervious material, thereby keeping the main fill some distance from the structure, the additional settlement load beneath the structure caused by the dike is less than 60 lb per sq ft. Even with this additional load, the net settlement load is still negative. Since the foundation clays have apparently been overconsolidated by alternate wetting and drying (see Laboratory Tests) and since the net settlement load is negative, there should be little or no settlement following construction as a result of the structure loads, if all loose and disturbed material is removed from the bottom of the excavation before the filter is placed. Expansion of the foundation due to unloading by the excavation and reconsolidation under the structure load will cause very little settlement, as the foundation materials are overconsolidated.

24. From the above analysis, it is considered that no piling will be necessary below the sector gates.

Lock gates

25. The loads of the lock gate section are summarized below:

	Load in Lb/Sq Ft
Construction condition	762
Water elev 6.0 (Gulf side) 0.0 (Grand Lake side) no uplift	1399
Water elev 6.0 (Gulf side) 0.0 (Grand Lake side) uplift to 0.0	555
Weight of excavated soil	830
Net settlement load	Negative

The load caused by the dike will be less than 120 lb per sq ft beneath the lock gate structure. A fill to be placed north of the dike and west of the lock channel to be used for the caretaker's residence will add about 25 lb per sq ft to the settlement load beneath the lock gate. The net settlement load is still negative. If any soft Recent material found beneath the base of the lock gate is replaced with suitable compacted backfill, and if all loose disturbed soil is removed from the bottom of the excavation, there should be little or no settlement following construction due to the structure loads.

26. No piling is considered necessary beneath the lock gate.

Underseepage

27. Underseepage as a result of differential head against the gates (6 ft) is of little significance, because of the very low permeability of the foundation material. However, as it will be necessary to dewater the gate structures in the event repairs are necessary to

the gates, uplift beneath the structure must be considered. If no means of pressure relief is provided, the uplift might exceed the weight of the structure. Therefore, a filter blanket with a collector pipe has been placed beneath the base slab (see plate 4) so that no differential in head can exist between the bottom and top of the slab.

28. The filter blanket is composed of two layers; 6 in. of gravel B beneath the concrete stabilization course, and 6 in. of filter sand A beneath gravel B. A third filter material, gravel C, should be placed around the collector pipe. This filter should have a minimum thickness of 4 in. measured from the outer circumference of the collector pipe. The gradations of these materials are shown on plate 20. The specifications for no. 4 to 3/4-in. concrete aggregate satisfy the limits set for gravel C, and fine concrete aggregate may be used as filter sand A. The collector pipe is 4-in. perforated clay pipe (1/2-in. holes). The outlet for the collector pipe will be brought up in the wall of the structure and discharges into the structure behind the gate at elev -13. The outer limit of the filter blanket shall not be closer than 10 ft to the edge of the structure. The layer of filter sand A should be compacted by 4 to 6 coverages of a tractor, as this material might bulk and cause settlement of the structure if placed in a loose condition.

29. In order that a check may be made on the performance of the filter during operations, it is recommended that two piezometers be installed under each gate. The location of the piezometers is shown on plate 4 and details of the piezometers on plate 19. The 'A'

piezometers are to measure the hydrostatic pressure in the foundation material immediately below the filter. The 'B' piezometers are to measure the hydrostatic pressures in the filter and to give an indication as to the head loss in gravels B and C and the collector pipe. The screens for the 'A' piezometers should be 12-in. lengths of 1-1/4-in. all-brass well point, such as Clayton-Mark, with No. 30 slots. The screen should be placed vertically in a 12-in. diameter hole about 4 ft deep, with the center of the screen about 2-1/2 ft. below the bottom of filter sand A. The hole should be backfilled with filter sand A except for the 12 in. immediately below the filter. In this 12 in. a sand-bentonite seal should be placed around the vertical piezometer pipe. The riser pipe should be 1/2-in. brass pipe, so as to reduce the time lag in the piezometer readings. The 'B' piezometers should have a 12-in. screen placed horizontally in filter sand A. For the 'B' piezometers, 1-1/4-in. brass riser pipe may be used. The pipes from the piezometers to the riser pipes should be placed on a slope of about 1 on 15 so as to allow any air in the piezometers to escape to the riser pipes. The riser pipes should be brought up in the wall of the structure and capped just above the top of the wall. The top of each piezometer should be permanently stencilled with the piezometer number.

30. When the structure is dewatered, the maximum permissible difference between the 'A' piezometer reading and the water elevation in the gate is 5.0 ft. This value provides a factor of safety of 1.3 against general uplifting of the structure.

Riprap

31. The purpose of gravel blankets beneath riprap is to retain the foundation material, and the gravel must be of such grading that it will not pass through the voids of the riprap. Based on these considerations, it is recommended that the gravel blanket beneath the riprap in the vicinity of the structure be as follows:

- a. A 7-in. layer of gravel F immediately below the riprap.
- b. A 5-in. layer of filter sand AA beneath gravel F and on the foundation soils.

The recommended limiting gradations of these filter materials are shown on plate 21. The standard concrete sand specifications as shown by the sand symbol fall within the limits of the filter sand AA, and No. 4 to 3/4-in. concrete aggregate satisfies the requirements for gravel F. However, it is pointed out that the concrete gravel represents about the minimum possible size of gravel F. Should it be used beneath the riprap, it would be desirable either to chink the hand-placed riprap or place the spalls from the riprap on top of the gravel blanket before placement of the coarser stone.

Spoil Banks

32. It was indicated by the Mississippi River Commission Design Section that most of the excavated material would be used as fill for the caretaker's residence. This fill may be placed adjacent to the lock portion of the channel to provide a continuous slope of 1 on 3 from elev +6 to elev -10. However, the portion of the fill composed of the soft Recent top strata should be placed some distance from the

excavation, and only material from the firmer Pleistocene formation should be placed near the channel. No spoil should be placed closer than 50 ft to the excavation for the control structure, and the height of such spoil should not be greater than 5 ft with a slope of 1 on 3.

Fender Piles

33. An axial load of 15 tons per pile and an average pile diameter of 10 in. were assumed for the design of the fender piles. For a factor of safety of 2.0, the required penetration is 24 ft for friction piles, assuming that the friction developed is equal to the cohesion of the material in which the pile is driven. This penetration below the bottom of the channel at the lock gate, elev -10, gives a pile length of roughly 40 ft if the piles are to be cut off at elev +6.0. This penetration should be satisfactory for any fender piles or dolphins near the control structure.

Conclusions

34. The following conclusions are made:

- a. Slopes of 1 on 3 for the construction excavation for the structure are satisfactory.
- b. Slopes of 1 on 2 are satisfactory for the dredged channel.
- c. The foundation beneath the structures has adequate bearing capacity to support the structures. No piling is considered necessary beneath the control or lock gate structures.
- d. In the event the Recent top stratum is found to extend below a portion of the base of the structures, it should be removed and replaced with compacted material.

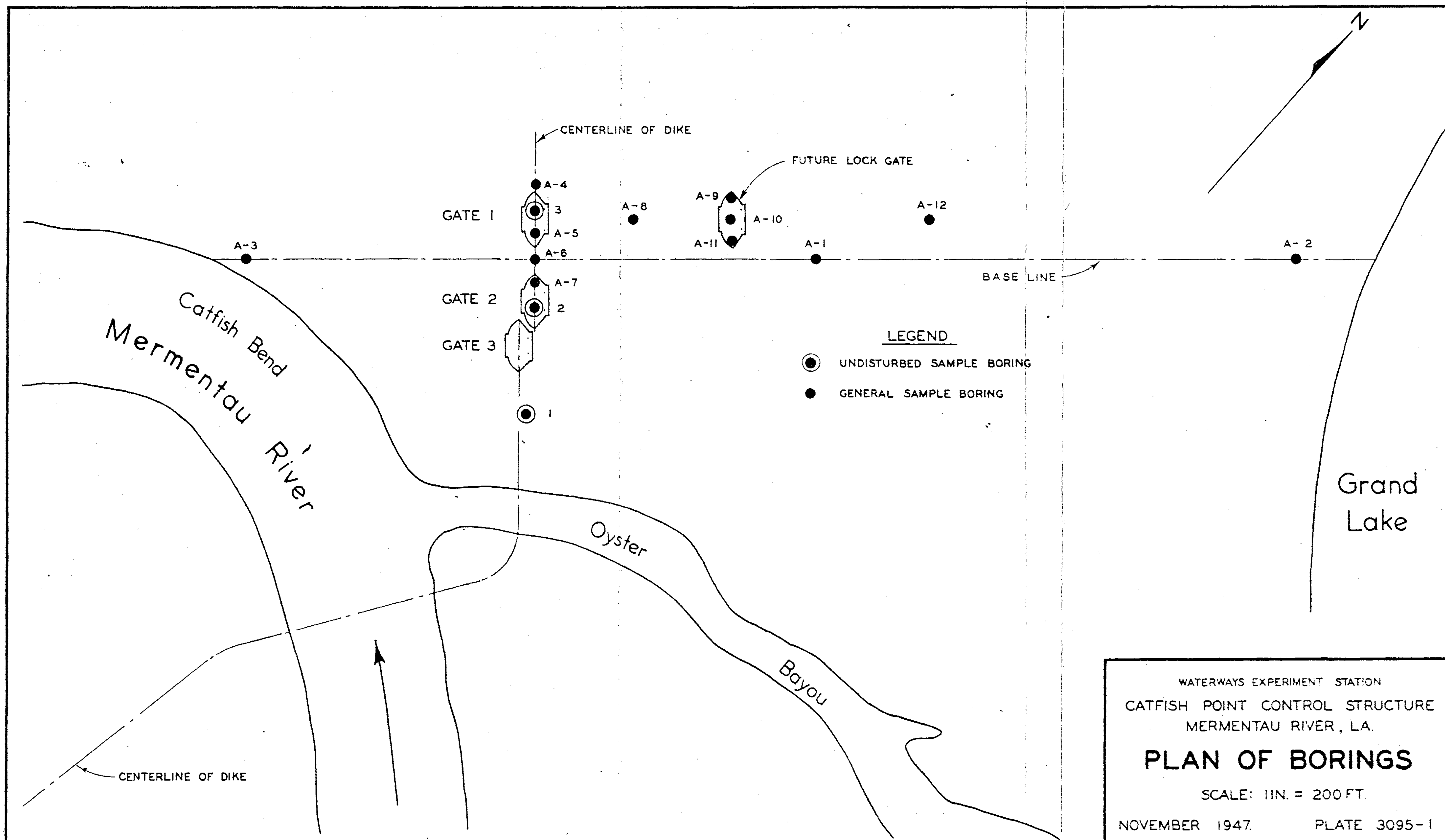
e. Little or no settlement is expected following construction due to structure load imposed on the foundation.

f. Filter blankets should be placed under the structures to prevent excessive uplift when the gates are dewatered for repair. Piezometers should be installed to check the operation of the filters.

g. Fill for the caretaker's residence may be placed to elev +6, giving a continuous 1 on 3 slope into the lock channel, if firm Pleistocene material is used for the fill adjacent to the channel.

h. No spoil should be placed closer than 50 ft to the excavation for the control structure, and the height of such spoil should be not greater than 5 ft with a slope of 1 on 3.

i. A penetration of about 24 ft will be satisfactory for the fender piles or dolphins.



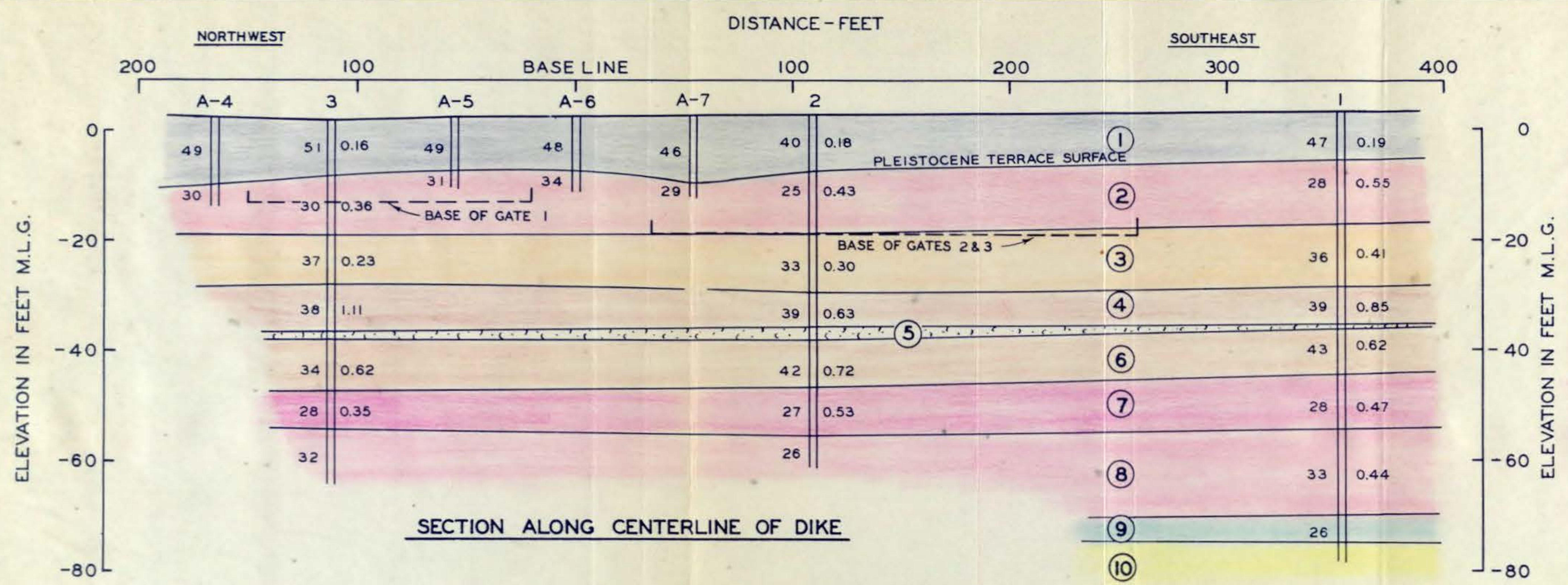
WATERWAYS EXPERIMENT STATION

CATFISH POINT CONTROL STRUCTURE
MERMENTAU RIVER, LA.

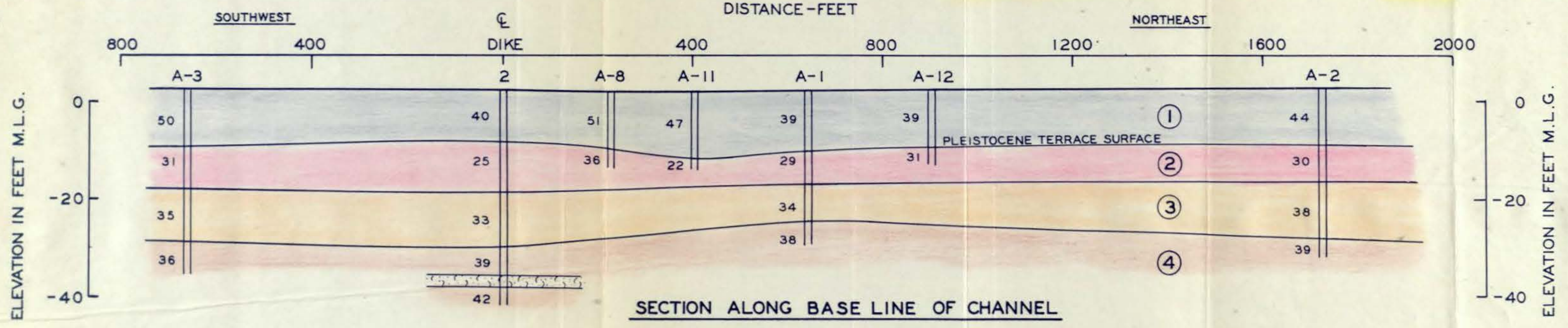
PLAN OF BORINGS

SCALE: 1 IN. = 200 FT.

NOVEMBER 1947 PLATE 3095-1



SECTION ALONG CENTERLINE OF DIKE



SECTION ALONG BASE LINE OF CHANNEL

Stratum	Description	Average[c] w - % Dry Wt.	Average[c] Cohesion T/Sq Ft	Design Cohesion T/Sq Ft
1	Silty clay, gray, soft (Recent material)	46	0.18	0.18
2	Silty clay with some sandy clay and clay sand, tan and gray, firm, some shells	29	0.44	0.40[d]
3	Clay and silty clay, red and gray, soft to firm	36	0.32	0.25[d]
4	Clay with many sandy silt lenses, tan and gray, stiff	38	0.91	0.65[e]
5	Shells, clay and sandy silt with shells	--	--	--
6	Clay and silty clay, gray, stiff, some shells	40	0.65	0.65
7	Clay silt, gray, soft to firm, some shells	28	0.47	0.46
8	Clay and silty clay, tan and gray, firm	30	0.44	0.46
9	Silty sand and clay sand, gray	26	--	--
10	Sand, gray, fine	--	--	--

Notes:

a - Figures to left of borings are average water contents of the strata at borings.

b - Figures to right of borings are average cohesive shear strengths of the strata at borings.

c - Averages based on all borings.

d - Design value average of borings 2 and 3.

e - Design value based on boring 2

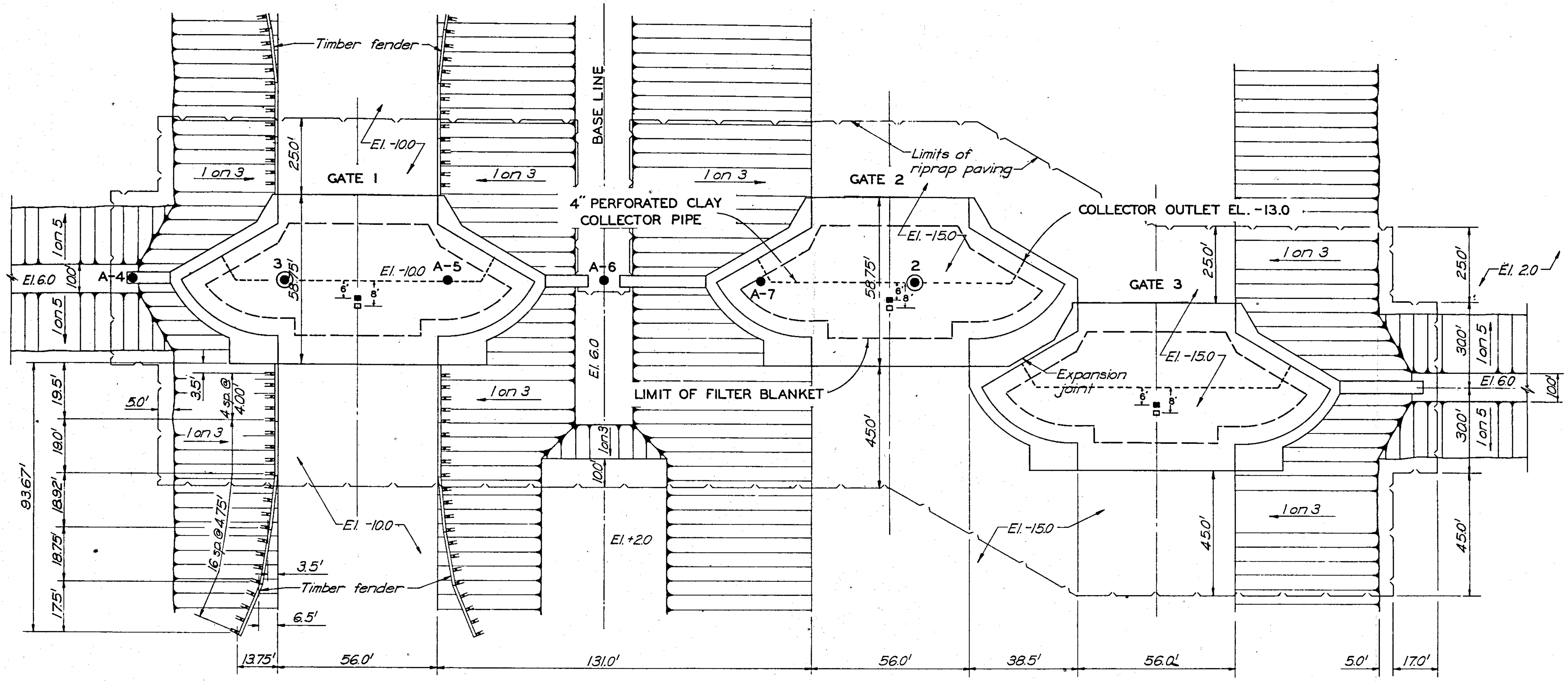
WATERWAYS EXPERIMENT STATION

CATFISH POINT CONTROL STRUCTURE
MERMENTAU RIVER, LA.

GENERALIZED SOIL PROFILE

SCALE: AS SHOWN

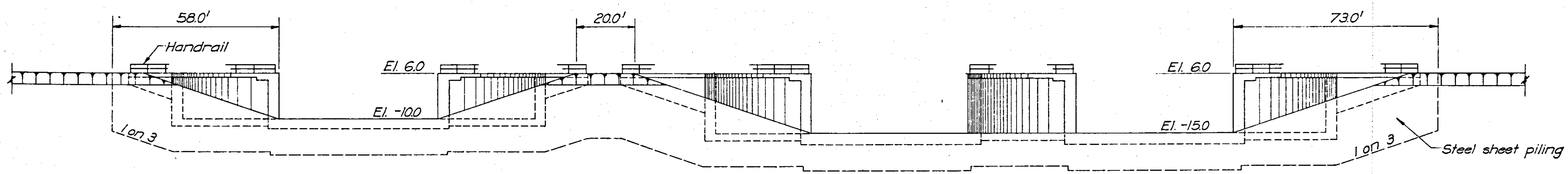
NOVEMBER 1947. PLATE 3095-3



PLAN

SCALE: 1 IN. = 20 FT.

20 FT. 0 20 40 60 80 100 FT.



ELEVATION

SCALE: 1 IN. = 20 FT.

LEGEND

- UNDISTURBED SAMPLE BORING
- GENERAL SAMPLE BORING
- "A" PIEZOMETER
- "B" PIEZOMETER

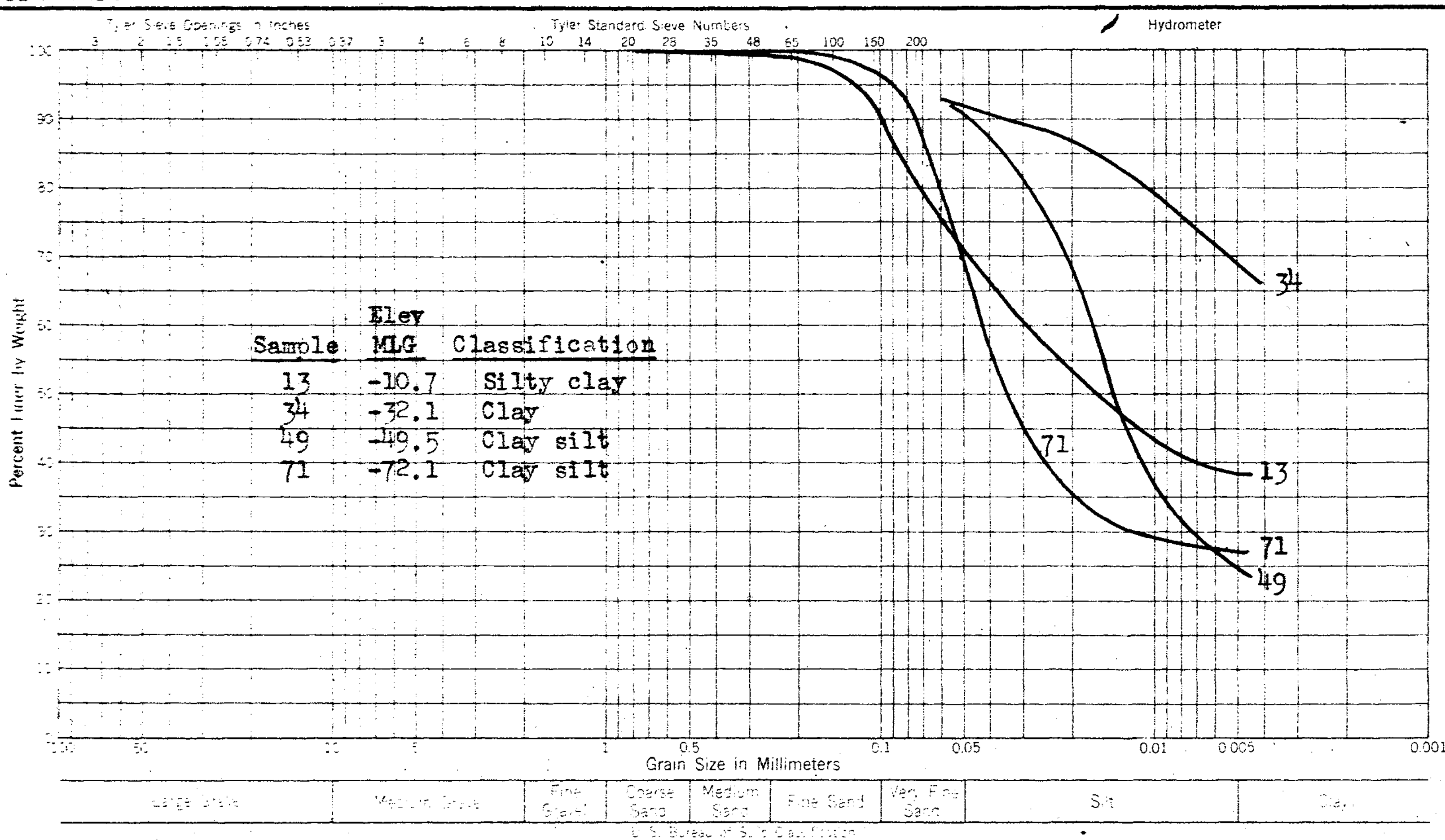
WATERWAYS EXPERIMENT STATION

CATFISH POINT CONTROL STRUCTURE
MERMENTAU RIVER, LA.

PLAN OF STRUCTURE

SCALE: AS SHOWN

NOVEMBER 1947. PLATE 3095-4



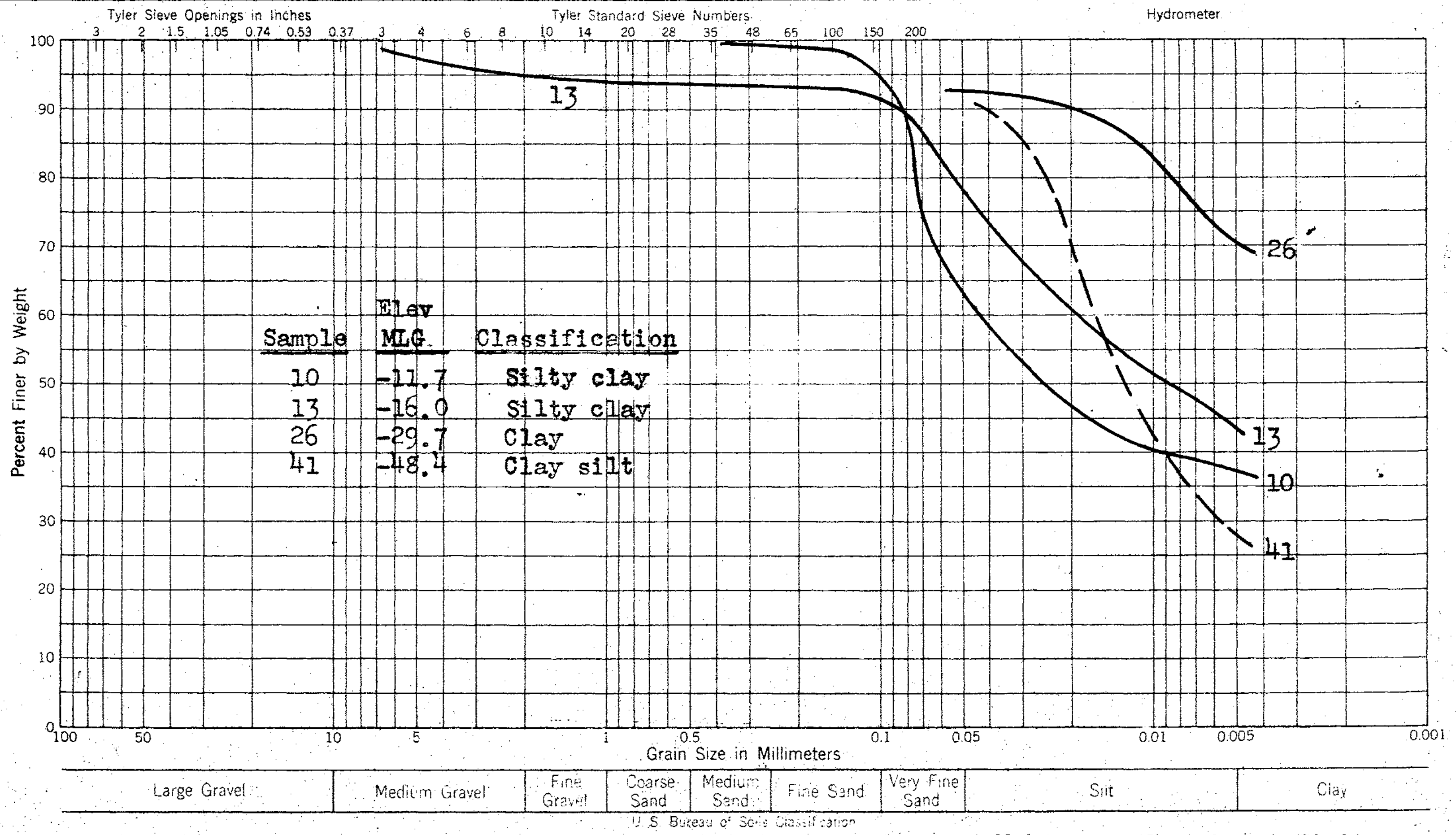
Waterways Experiment Station
Catfish Point Control Structure

GRAIN SIZE DISTRIBUTION CURVES

Boring 1

November 1947 Plate 3095-5

WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY



Sample	Elev MLG.	Classification
10	-11.7	Silty clay
13	-16.0	Silty clay
26	-29.7	Clay
41	-48.4	Clay silt

Waterways Experiment Station
Catfish Point Control Structure

GRAIN SIZE DISTRIBUTION CURVES

Boring 2

November 1947 Plate 3095-6

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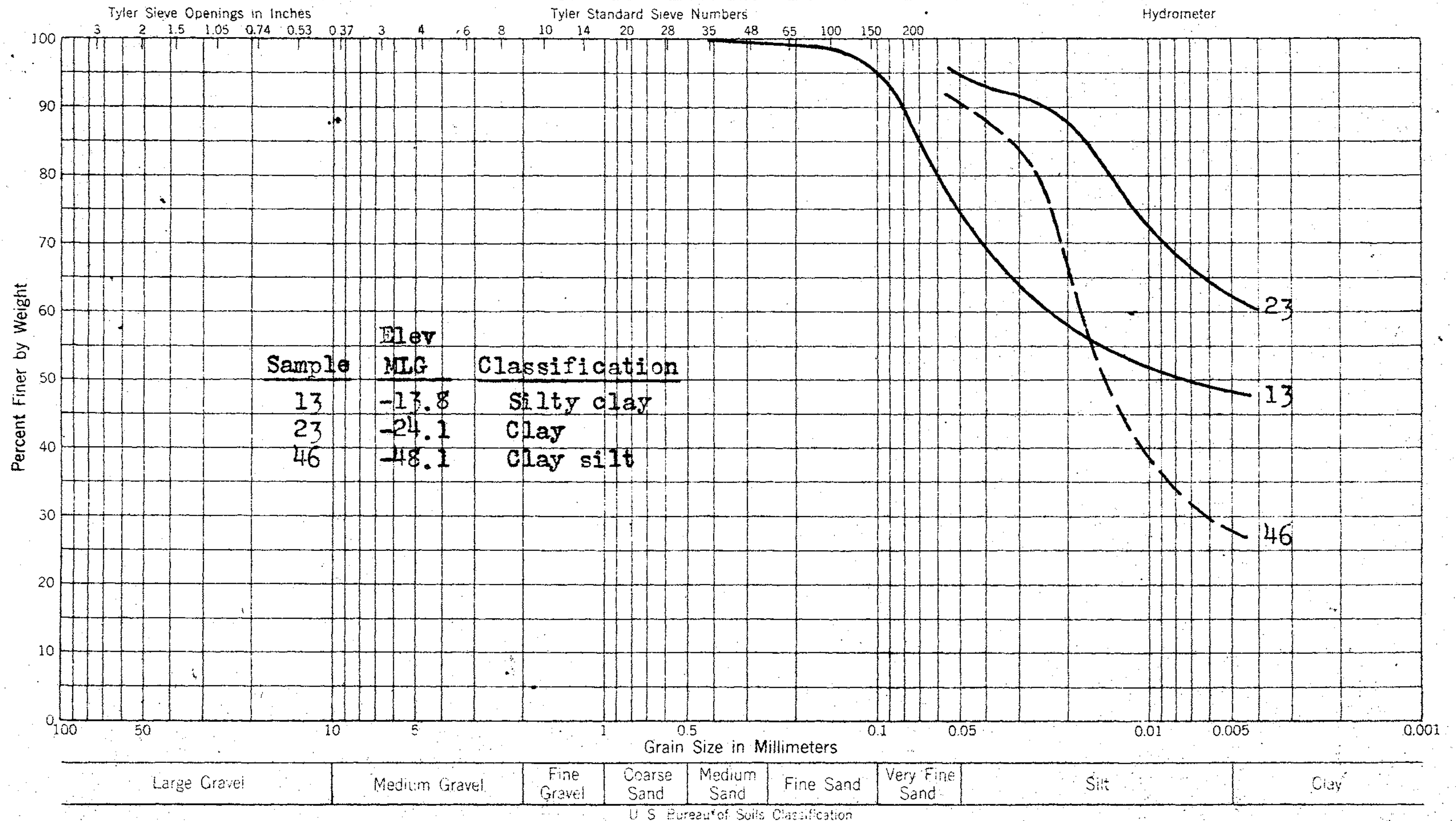
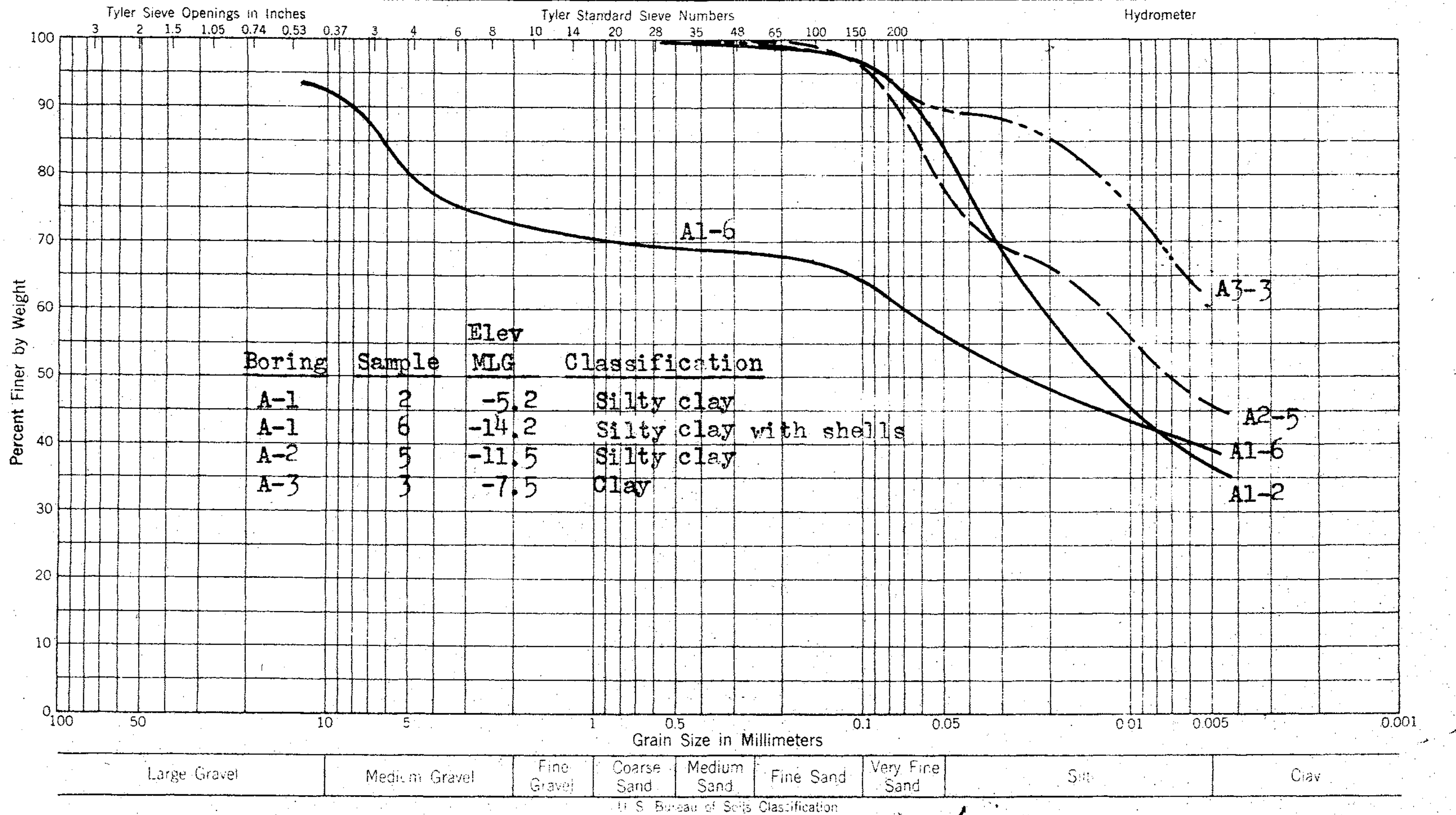


Waterways Experiment Station
Catfish Point Control Structure

GRAIN SIZE DISTRIBUTION CURVES
Boring 3
November 1947 Plate 3095-7

WAR DEPARTMENT

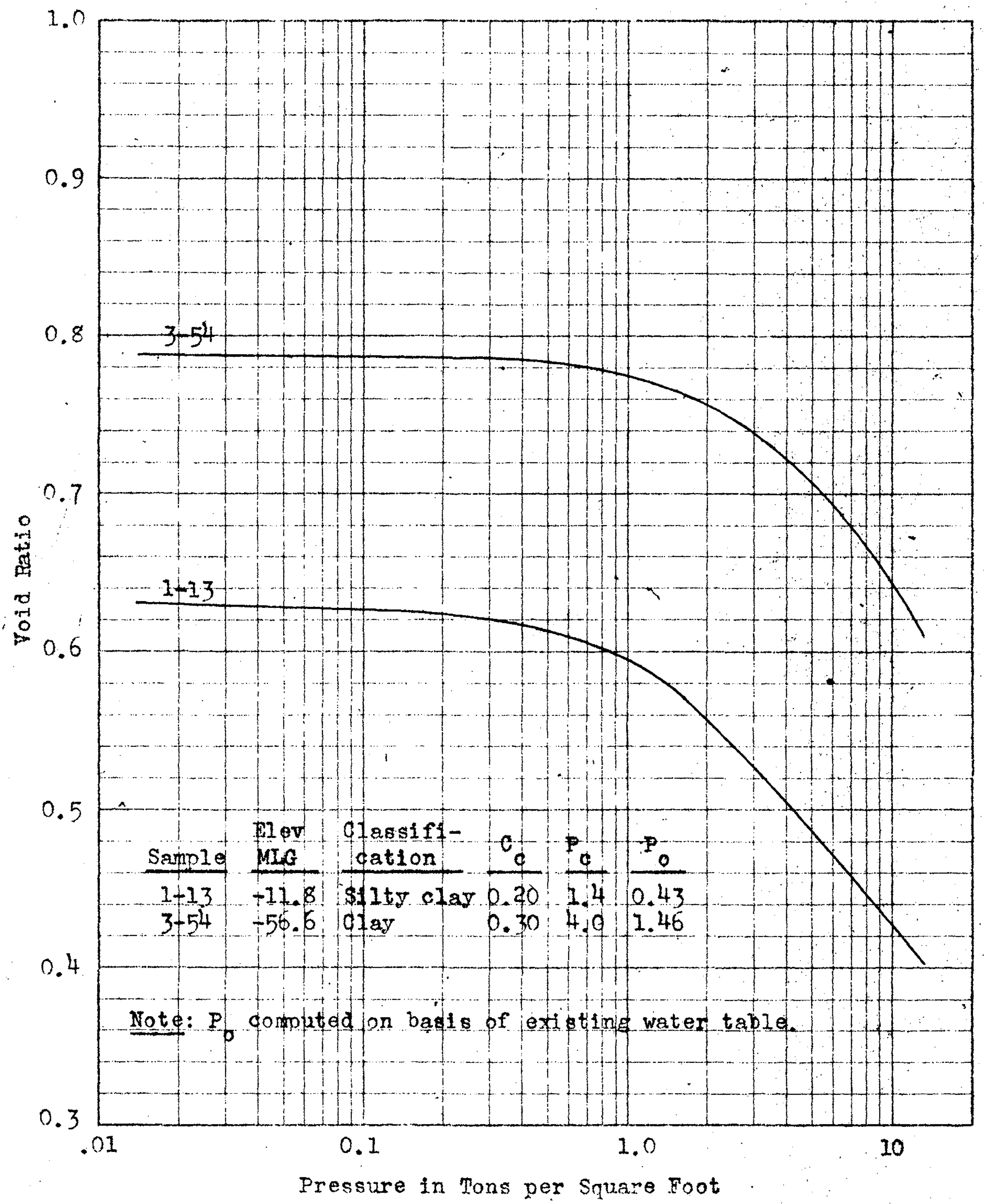
CORPS OF ENGINEERS U. S. ARMY



Waterways Experiment Station
Catfish Point Control Structure

GRAIN SIZE DISTRIBUTION CURVES
Borings A1, A2, and A3

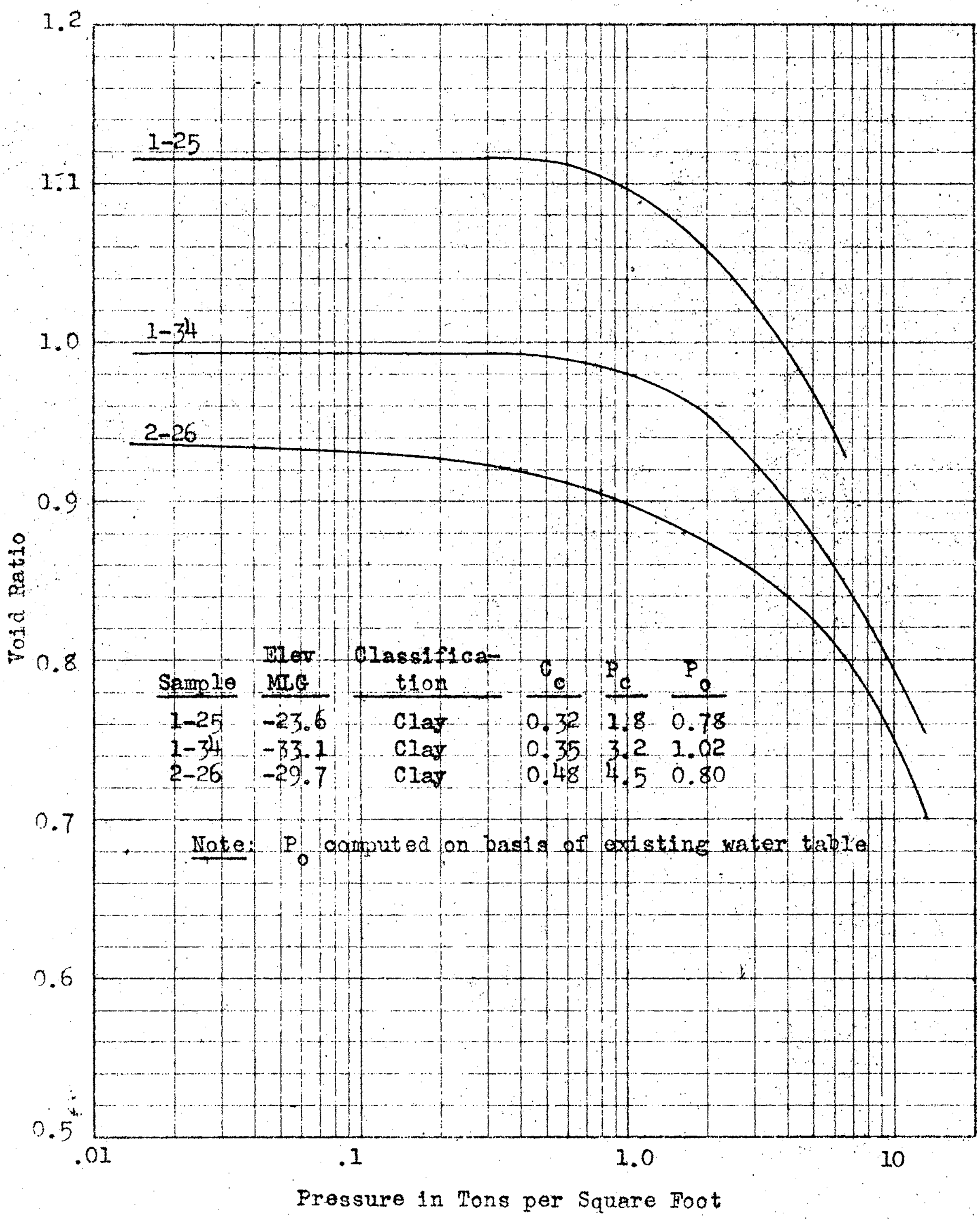
November 1947 Plate 3095-8



Waterways Experiment Station
Catfish Point Control Structure

PRESSURE-VOID RATIO CURVES

November 1947 Plate 3095-9



Sample	Elev MLG	Classification	C_c	P_c	P_o
1-25	-23.6	Clay	0.32	1.8	0.78
1-34	-33.1	Clay	0.35	3.2	1.02
2-26	-29.7	Clay	0.48	4.5	0.80

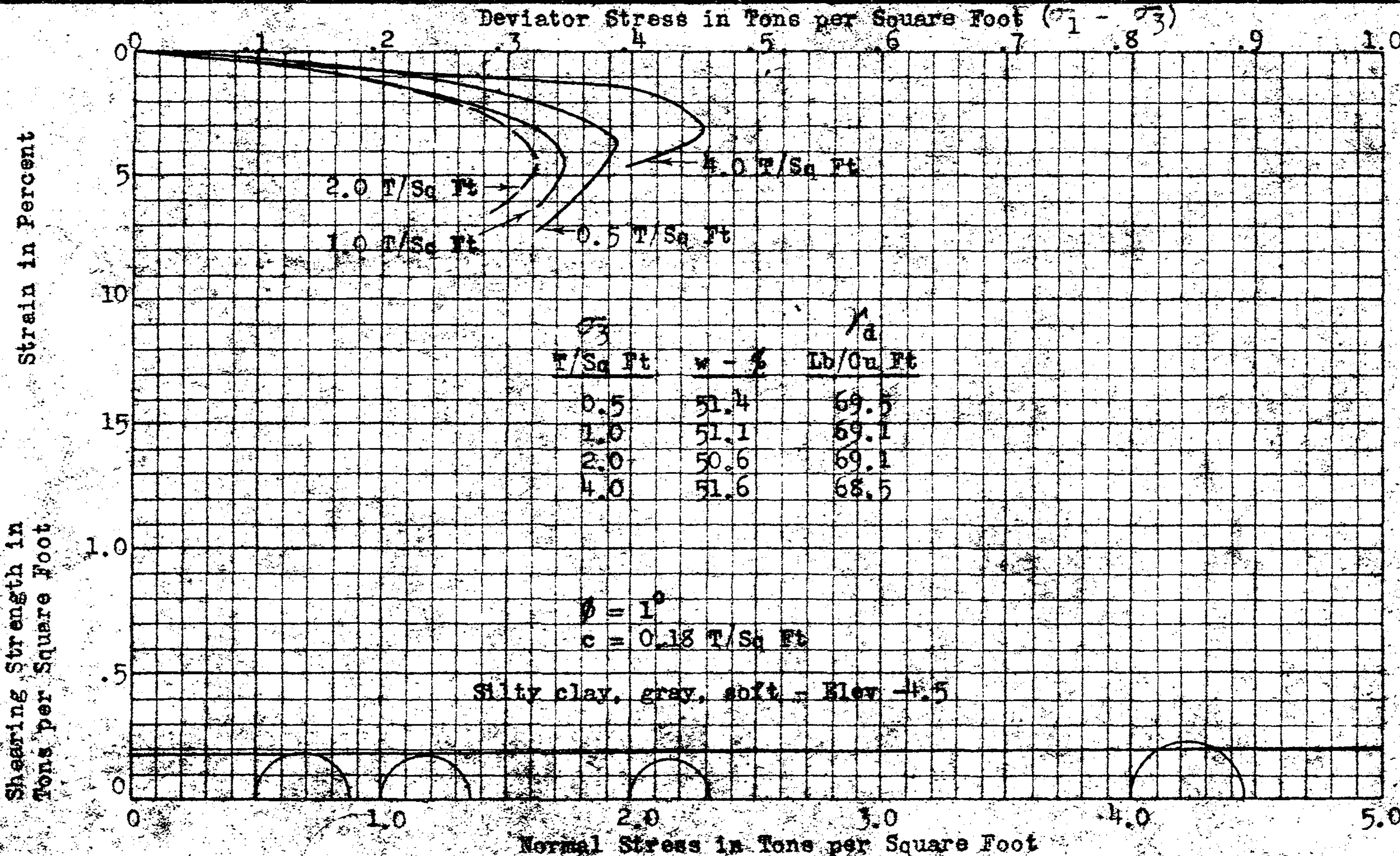
Note: P_o computed on basis of existing water table

Waterways Experiment Station
Catfish Point Control Structure

PRESSURE-VOID RATIO CURVES

November 1947 Plate 3095-10

WAR DEPARTMENT CORPS OF ENGINEERS, U.S. ARMY



σ_3 T/Sq Ft	w - %	γ_d Lb/Cu Ft
0.5	51.4	69.5
1.0	51.1	69.1
2.0	50.6	69.1
4.0	51.6	68.5

$\phi = 1^\circ$
c = 0.18 T/Sq Ft

Silty clay, gray, soft - Elev -4.5

Waterways Experiment Station
Catfish Point Control Structure
QUICK TRIAXIAL COMPRESSION TEST
Boring 1, Sample 7
November 1947 Plate 3095-11

WAR DEPARTMENT

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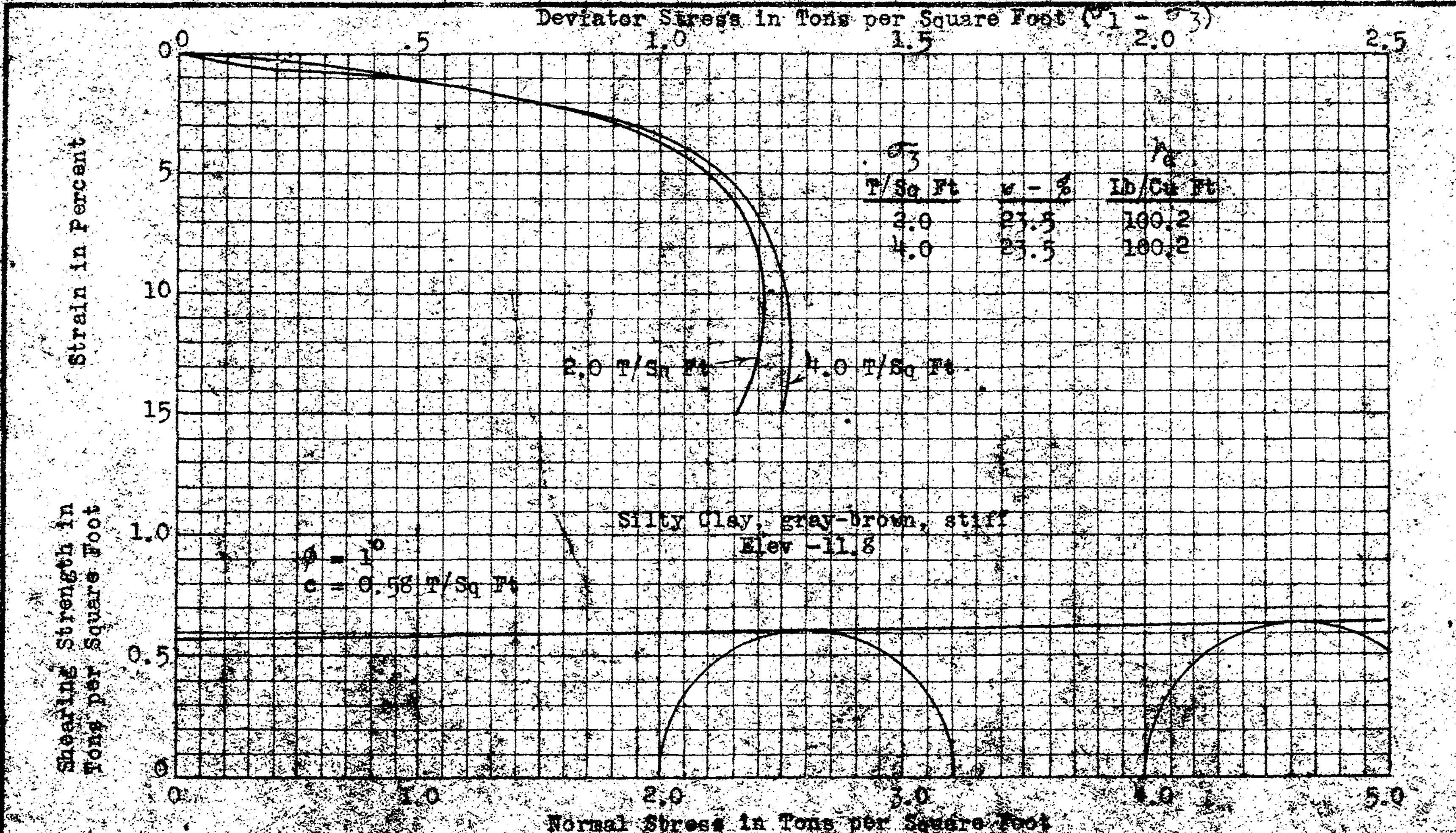
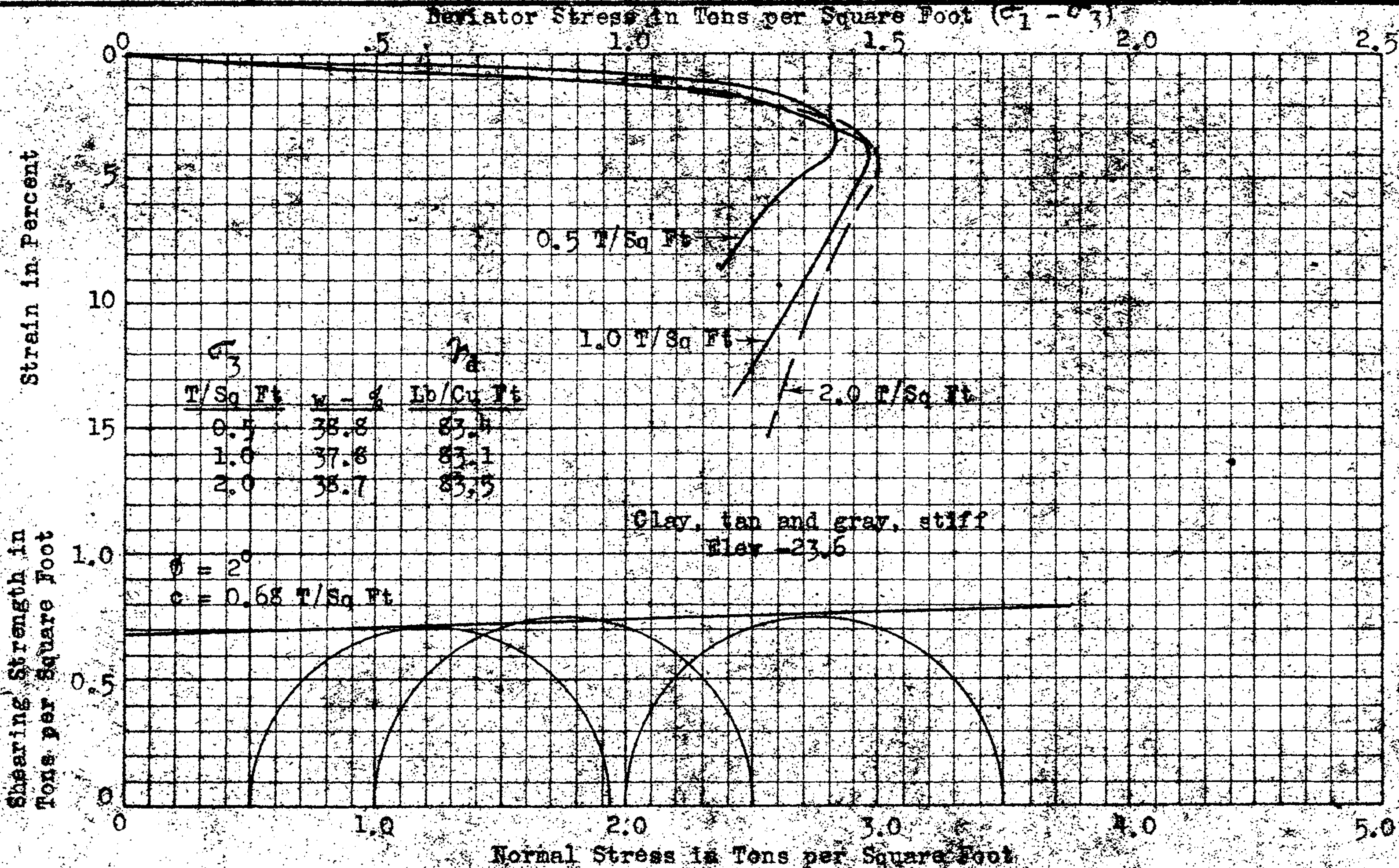
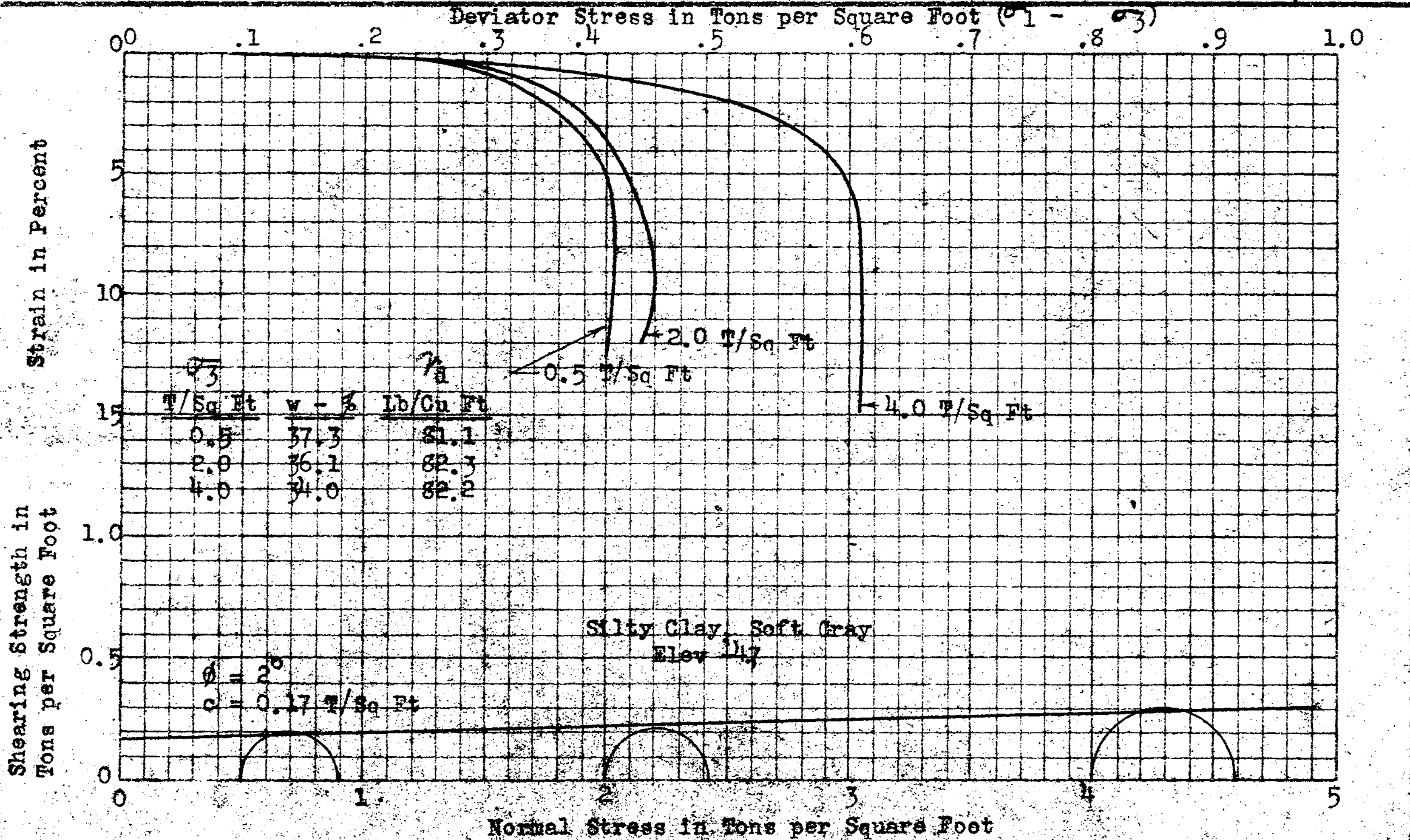


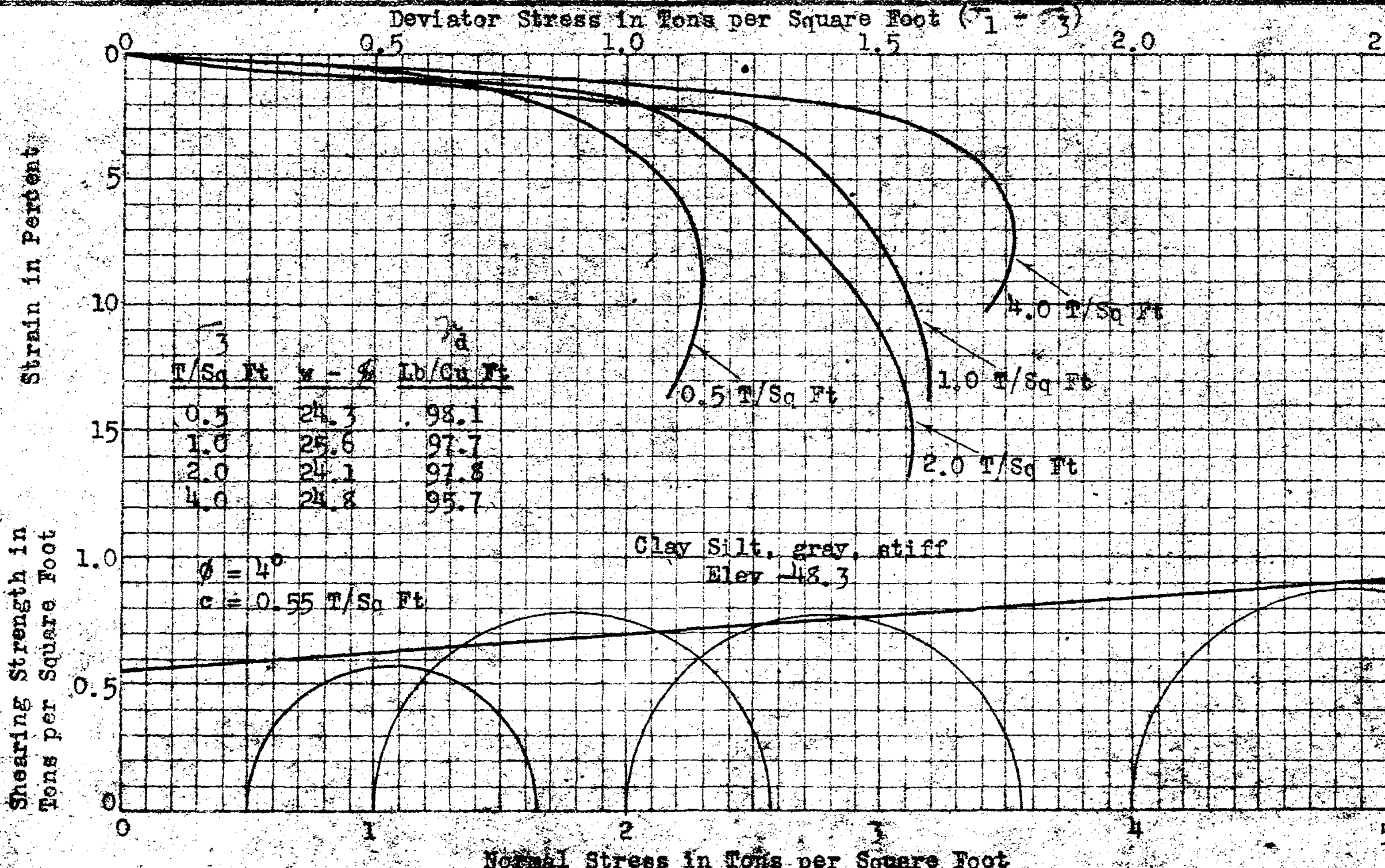
Waterways Experiment Station
Catfish Point Control Structure
QUICK TRIAXIAL COMPRESSION TEST
Boring 1, Sample 13
November 1947 Plate 3095-12



Waterways Experiment Station
Catfish Point Control Structure
QUICK TRIAXIAL COMPRESSION TEST
Boring 1, Sample 25
November 1947 Plate 3095-13

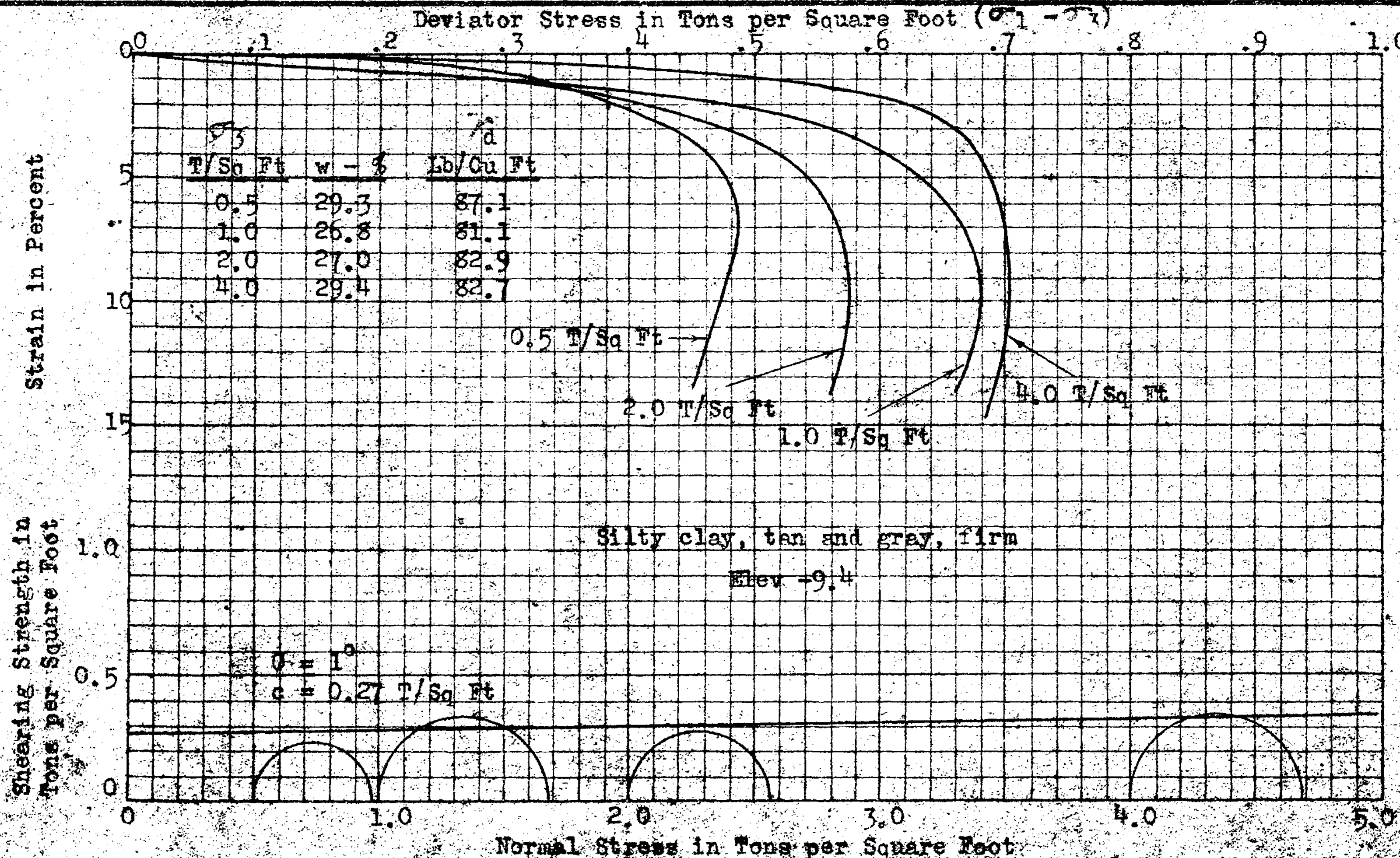


Waterways Experiment Station
Catfish Point Control Structure
QUICK TRIAXIAL COMPRESSION TEST
Boring 2 - Sample 6
November 1947 Plate 3095-14

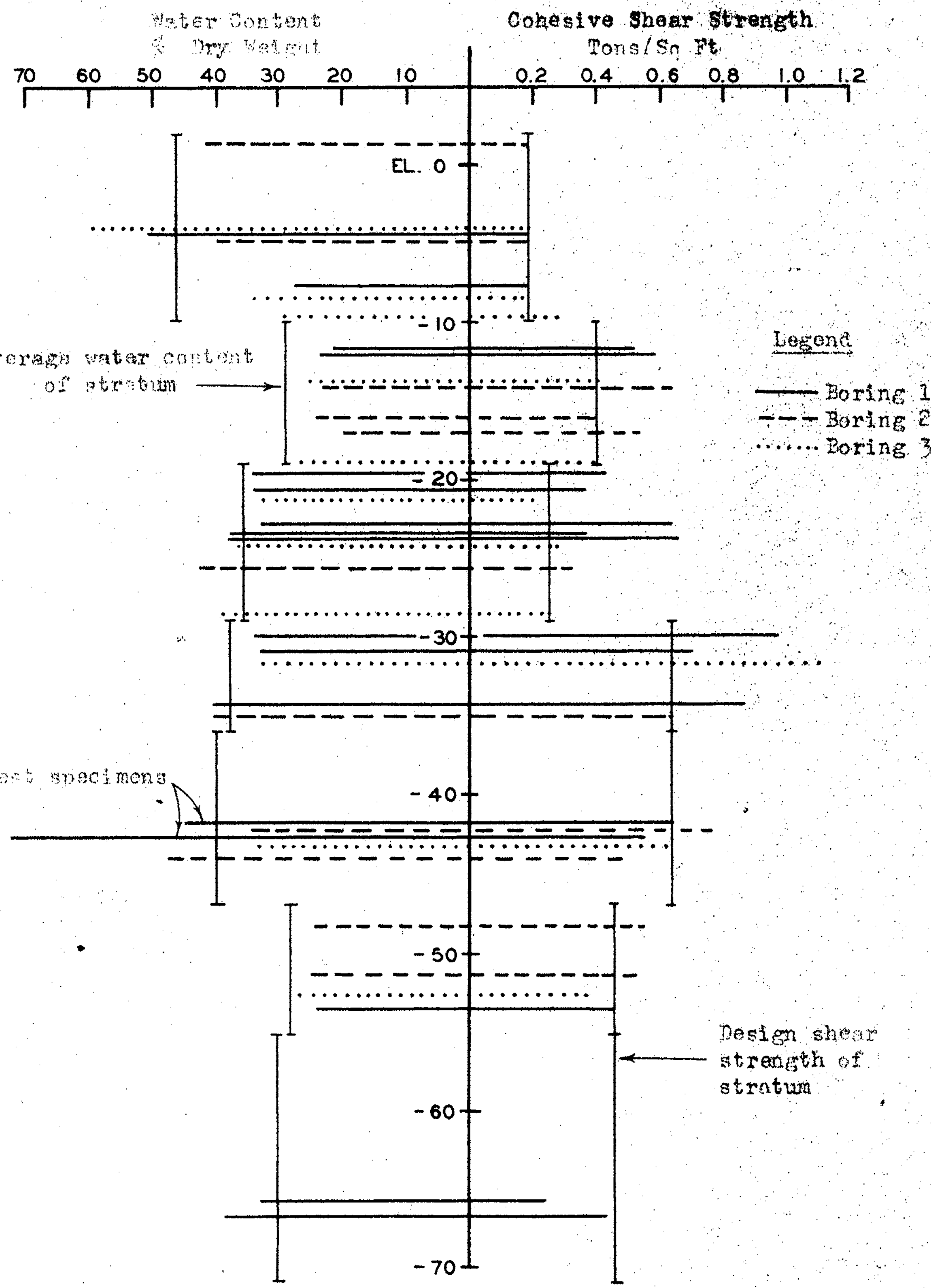


σ_3 T/Sq Ft	w - %	γ_d Lb/Cu Ft
0.5	24.3	98.1
1.0	25.6	97.7
2.0	24.1	97.8
4.0	24.8	95.7

Waterways Experiment Station
Catfish Point Control Structure
QUICK TRIAXIAL COMPRESSION TEST
Boring 2 - Sample 41
November 1947 Plate 3095-15



Waterways Experiment Station
Catfish Point Control Structure
QUICK TRIAXIAL COMPRESSION TEST
Boring 3, Sample 9
November 1947 Plate 3095-16



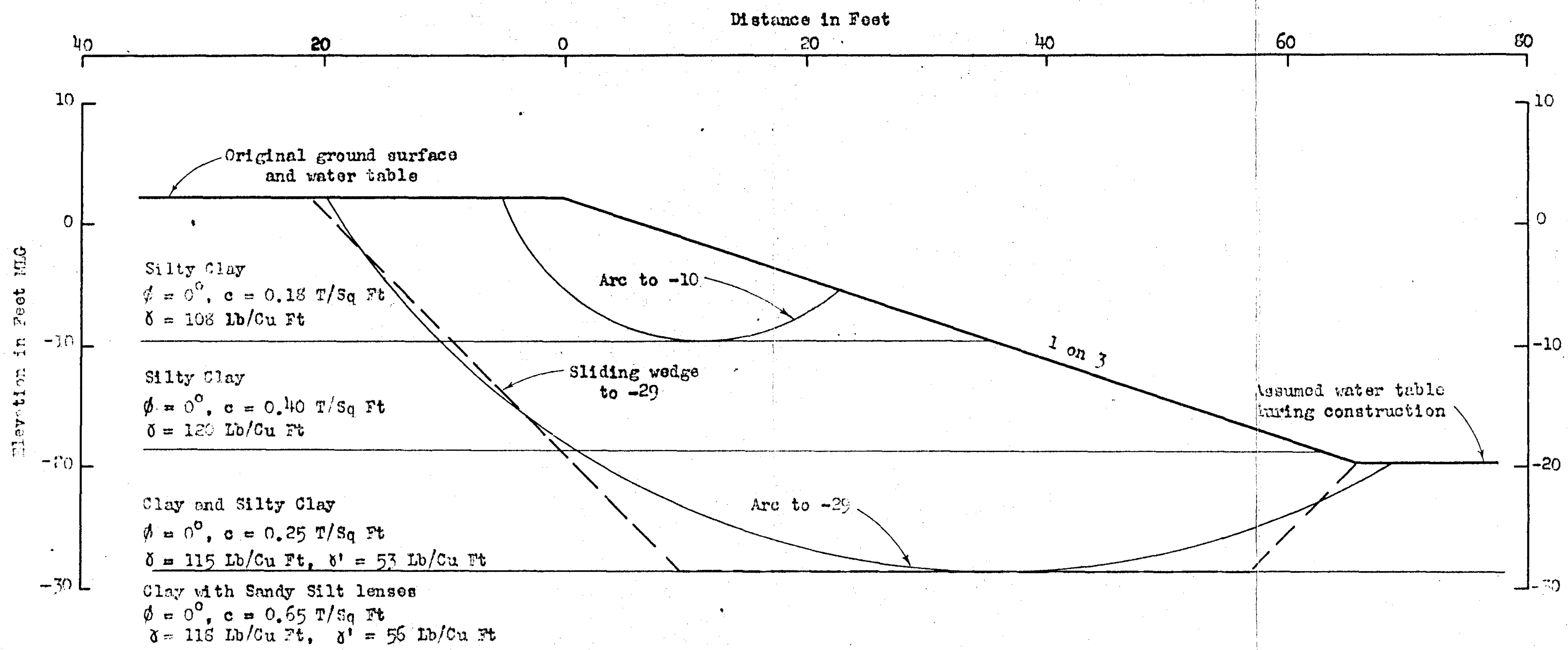
Waterways Experiment Station
Catfish Point Control Structure

WATER CONTENTS AND COHESIVE SHEAR STRENGTH

Borings 1, 2, and 3

November 1947

Plate 3095-17



Analysis	Factor of Safety
Arc to -10	2.70
Arc to -29	1.50
Sliding wedge to -29	1.36

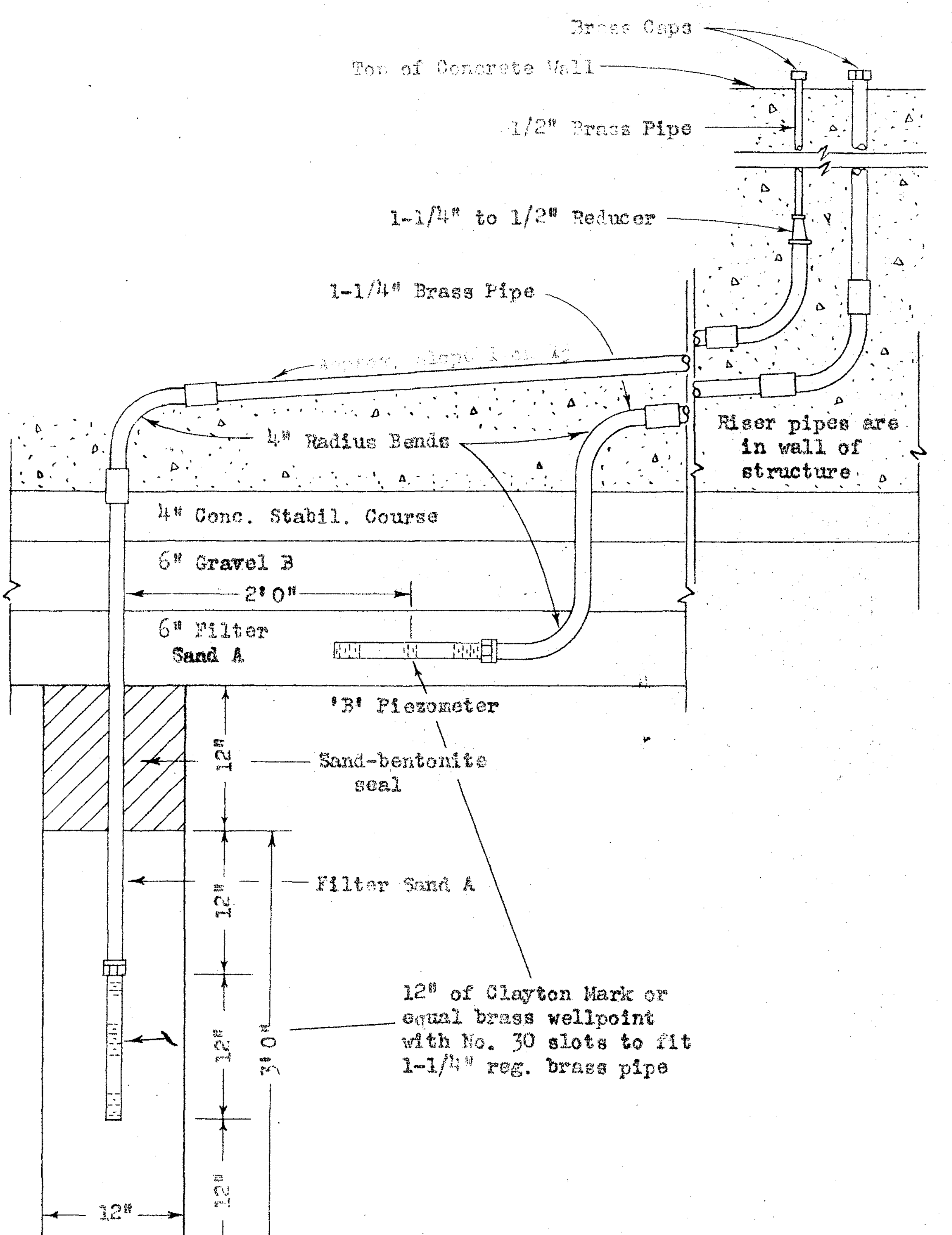
Waterways Experiment Station
Catfish Point Control Structure

CRITICAL FAILURE SURFACES - CONSTRUCTION SLOPES

Scale: 1" = 10'

November 1947

Plate 3095-18



'A' Piezometer

Waterways Experiment Station
Catfish Point Control Structure

PIEZOMETER DETAILS

Scale: 1" = 1'

November 1947 Plate 3095-19

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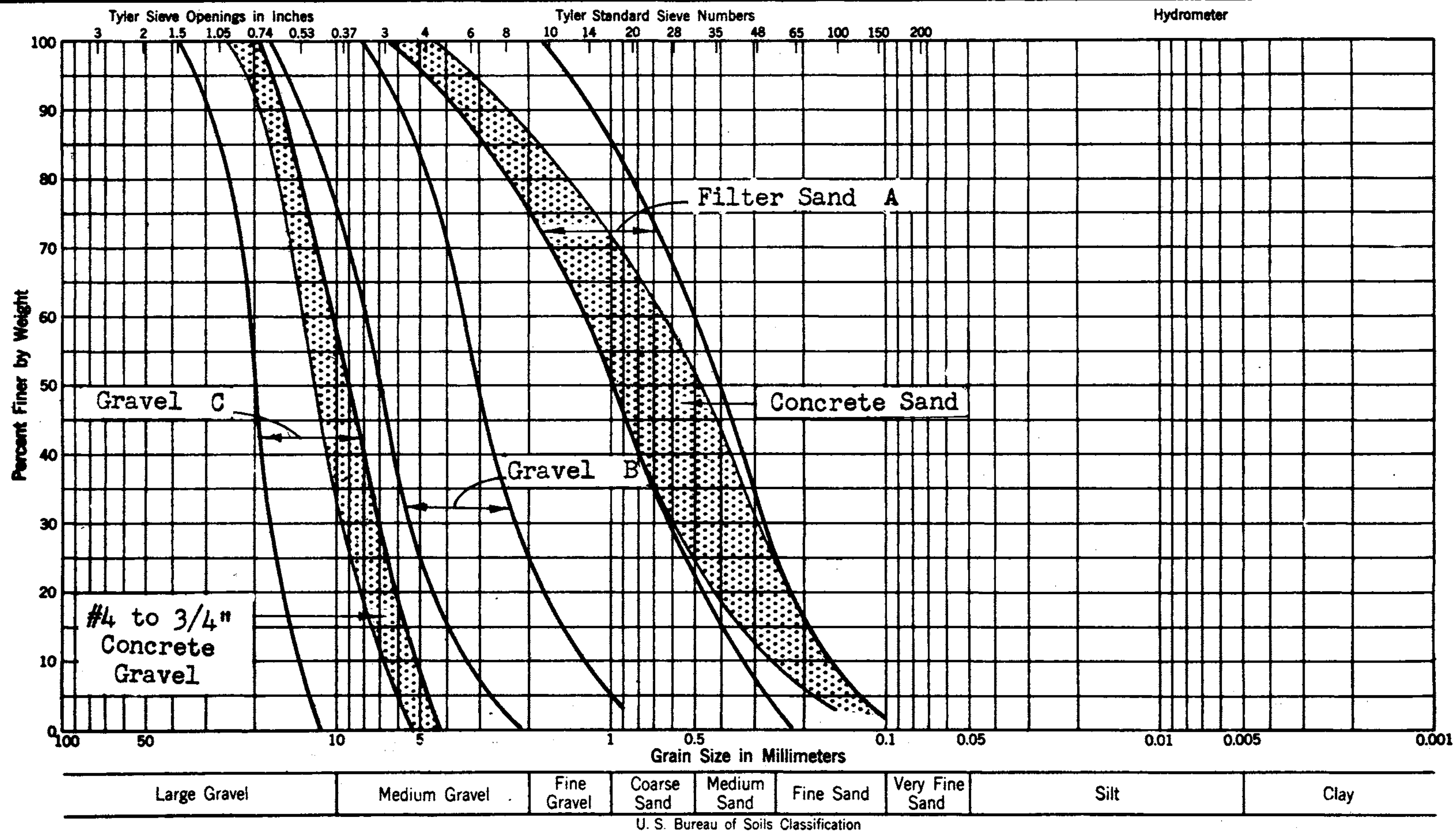

Waterways Experiment Station
Catfish Point Control Structure

FILTER MATERIALS

November 1947 Plate 3095-20

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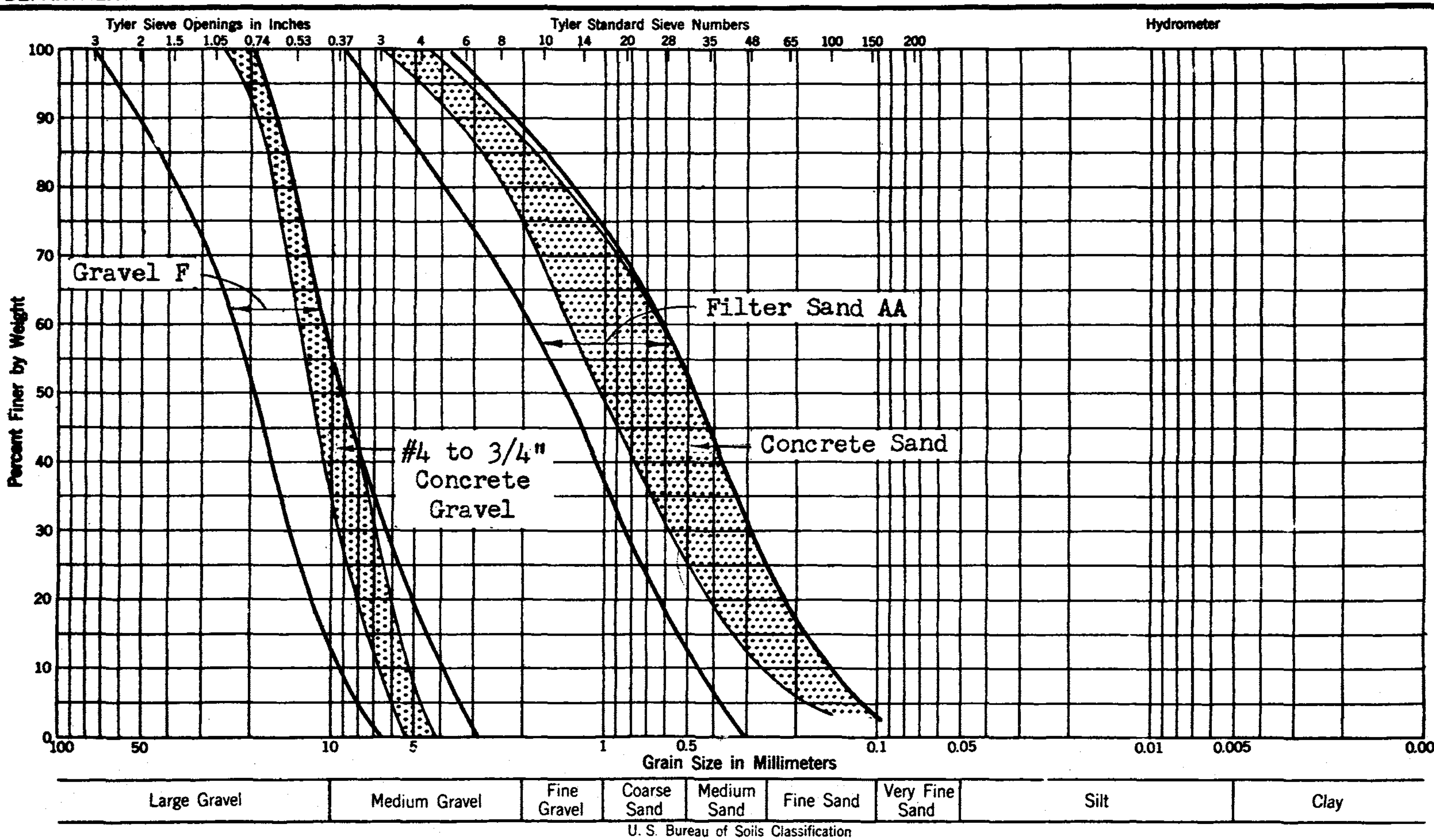

Waterways Experiment Station
Catfish Point Control Structure

GRAVELS BENEATH RIPRAP

November 1947 Plate 3095-21